AS-BUILT FOUNDATION REPORT
AUXILIARY CONDUIT TUNNEL

PARK RIVER LOCAL PROTECTION

HARTFORD CONNECTICUT

VOLUME I

MAIN REPORT

DECEMBER 1982

TC423
.N43P236
1982
v.1
c.2

[illegible] Corps
of Engineers
New England Division

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE

READ INSTRUCTIONS BEFORE COMPLETING FORM

1. REPORT NUMBER	2. GOVT ACCESSION NO.	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) PARK RIVER LOCAL PROTECTION AS-BUILT FOUNDATION REPORT AUXILIARY CONDUIT TUNNEL		5. TYPE OF REPORT & PERIOD COVERED MAIN REPORT VOLUME I 6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) U. S. ARMY CORPS OF ENGINEERS NEW ENGLAND DIVISION		8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS DEPT. OF THE ARMY, CORPS OF ENGINEERS NEW ENGLAND DIVISION 424 TRAPELO ROAD, WALTHAM, MASS. 02254		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS		12. REPORT DATE DECEMBER 1982 13. NUMBER OF PAGES 41 PAGES 48 PLATES
14. MONITORING AGENCY NAME & ADDRESS(if different from Controlling Office)		15. SECURITY CLASS. (of this report) UNCLASSIFIED 15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

APPROVED FOR PUBLIC RELEASE: DISTRIBUTION UNLIMITED

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

PARK RIVER AUXILIARY TUNNEL
AS-BUILT FOUNDATION REPORT

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Tunnel
Precast Liner
Flood Control
Inverted Syphon

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The report documents the construction of an inverted syphon shaped flood control auxiliary conduit using a precast concrete tunnel lining. The use of this construction technique is the first time it has been used by the Corps of Engineers below the ground water surface in an urban environment. The construction history including time and cost is summarized in charts and graphs with a narrative description of construction procedures and documentation.

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PARK RIVER LOCAL PROTECTION AUXILIARY CONDUIT TUNNEL

AS-BUILT FOUNDATION REPORT

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PARK RIVER LOCAL PROTECTION PROJECT

AUXILIARY CONDUIT TUNNEL

CONSTRUCTION FOUNDATION REPORT

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B. Subsidence Survey Report

PARK RIVER LOCAL PROTECTION PROJECT

AUXILIARY CONDUIT TUNNEL

CONSTRUCTION FOUNDATION REPORT

1. LOCATION AND DESCRIPTION OF PROJECT

The Park River Local Protection Project is located in the City of Hartford, Connecticut and initially consisted of sections of a twin-rectangular reinforced concrete conduit built in 1944 to inclose the lower 5,600 feet of Park River. Its purpose is to protect the low level built-up area of the city from the Connecticut River backwater and from floods caused by runoff from the Park River Basin. In 1968 the City of Hartford through the Greater Hartford Flood Commission working with the State of Connecticut, completed four additional sections of twin-rectangular conduti in conjunction with the construction of Interstate Highway 84. The original Park River Conduit was designed for a discharge capacity of 18,000 cfs. A restudy indicated that a greater design storm flow than previously used could occur due to additional development in the basin, and could produce a project design flood of 30,000 cfs causing conditions which would overtop river banks and headwalls resulting in serious flooding of downtown Hartford. Completion of the open sections in the existing box conduit system would permit pressuring the entire conduit system thereby increasing the overall capacity.

Completion of the remaining four sections with a maximum capacity of 18,000 cfs would not provide project design flood protection for the City of Hartford. Alternatives investigated indicated that a 22 foot diameter auxiliary conduit would provide a high degree of flood protection in the densely populated and developed urban areas.

This report discusses the foundation conditions as they relate to the 9,100 foot long auxiliary conduit tunnel shaped as an inverted siphon up to 200 feet in depth below the city. The auxiliary tunnel is designed to carry 5,400 cfs of the box conduit overflow, transmitting flood waters from an upstream portion of the box conduit directly to the Connecticut River thus reducing flood damages and losses in the headpool areas. (See Plate 1)

2. CONSTRUCTION AUTHORITY

The Park River Local Protection Project was authorized by the Flood Control Act of 1968, Public Law 90-483 dated August 13, 1968 which reads in part as follows:

> "The project for flood protection on Park River, Connecticut is hereby authorized substantially in accordance with the recommendation of the Chief of Engineers in Senate Document No. 43, Ninetieth Congress, at an estimated cost of $30,000,000."

3. PURPOSE OF REPORT. The purpose of this report is to fulfill the requirement of Regulation ER 1110-1-1801 dated 14 January 1972 to prepare an as-built foundation report for all major and unique construction projects. The reason for preparing this report is to insure the preservation for future use of complete records of foundation conditions encountered during construction and of methods used to adapt structures to these conditions.

The most common uses to which foundation reports are put are (1) in determining the validity of claims made by construction contractors in connection with difficulties arising from alleged foundation conditions or from alleged changed conditions (2) in planning additional foundation treatment should the need arise after project completions, (3) in evaluating the cause of failure and in planning remedial action should failure or partial failure of a structure occur as a result of foundation deficiencies (4) for guidance in planning foundation explorations and in anticipating foundation problems for future comparable construction projects and (5) as part of the permanent collection of project engineering data required by Appendix 1 to ER 1110-2-100.

4. LOCATION OF STRUCTURE. The intake structure is located (see Plate 1) at the junction of box conduit sections 4, 5 and 7 and proceeds due east beneath Park Street and other portions of the City of Hartford, terminating at the westerly bank of the Connecticut River approximately 1,000 feet downstream of the box conduit confluence with the Connecticut River. The intake structure is located at coordinates N 148,803.936, E 162,998.896 proceeding on a 5,000 foot curve to the east until it intersects Park Street where upon it proceeds due east terminating at the outlet structure located at coordinates N 148,368.647, E 172,024.122. The tunnel was designed with an invert design grade of - 95.07 MSL at the intake sloping downgrade at 0.5778% to an invert elevation of -147.36 at the outlet portal. Location plan and as built tunnel configuration is shown on Plate 1.

5. CONTRACTS AND CONTRACT SUPERVISION

Prime contractor for the project was Roger J. Au & Son, Inc., P.O. Box 1488, Mansfield, OHIO 44901. Sub-contractors were as follows:

John V. Dinan Associates Inc.
Consulting Engineer and Seismologists
2125 Center Avenue
Fort Lee, NJ 07024

San-Vel Concrete Corp.
Prestress Divisions
P.O. Box 385
Littleton, MA 01460

Geotechnical Engineers Inc.
1017 Main Street
Winchester, MA 01890

The Robbins Company
650 S. Orcas Street
Seattle, WASH 08108

Goldberg Zoino Dunnicliff Associates Inc.
Geotechnical Consultants
30 Tower Road
Newton Upper Falls, MA 02164

Richland Engineering LTD
Consulting Engineers
2770 Lexington Ave.
P. O. Box 3145
Mansfield, OHIO 44904

The Government was responsible for overall contract administration, supervision and inspection. During the peak workload employment reached 143 people, of which 13 were Government personnel. Significant construction events are shown on Table 1. The bid price on 23 June 1977 for the work was $23,248,185 and was completed as of 2 July 1981 at a cost of $24,457,493.

TABLE 1

PARK RIVER LOCAL PROTECTION

AUXILIARY CONDUIT TUNNEL

SIGNIFICANT CONSTRUCTION EVENTS

FEATURE	DATE
Contract Award	23 June 1977
Notice to Proceed	27 Sept 1977
Construction Start (Outlet Shaft)	1 Feb 1978
TBM Fabrication Chamber Completed	3 May 1979
TBM Operational	13 Jul 1979
Intake Shaft Started	6 Oct 1979
Intake Shaft Completed	4 June 1980
TBM Holed Thru (Intake Shaft)	3 Jul 1980
Outlet Shaft Completed	21 Aug 1981
Tunnel Completed and Operational	2 Jul 1981
Final Payment	19 Jul 1982

B. FOUNDATION EXPLORATION

6. INVESTIGATION PRIOR TO CONSTRUCTION

a. General. Explorations consisted of core borings, various tests within the boreholes and a seismic survey. Tests in boreholes included borehole photographs, pressure testing, piezometers, observation wells and pump tests.

(1) Rock core from 29 borings were used to determine the tunnel geology. Eighteen were M-series chrome barrel NX diameter core and eleven were M-series 4-inch diameter. (See plates 2-16 for location) Ten boreholes do not reach tunnel grade because they were drilled for a previous design in which the tunnel was designed for a higher invert elevation. Rock cores were record photographed in the laboratory. All cores were photographed in the field immediately upon removal from the core barrel for future use during contract bidding. Six wash borings were made to determine the bedrock surface configuration at the outlet portal and one boring was made to a depth of 155.5 feet (OC-1) to determine the in-situ rock stresses by the overcoring method. (Appendix A)

(2) Borehole photography was conducted on 22 boreholes in rock to determine the in-situ rock structure. All of the rock structure was observed for joint structure frequency and orientation. Typical joint rosettes and polar diagrams are plotted on Plates 2 thru 16.

(3) Pressure testing was conducted on all borings at 5 foot zones using air inflatable or mechanical packers. Graphic representation of pressure test results are shown on Plates 2 thru 16. Where artesian flows are encountered, pressure testing was restricted.

(4) Observation wells were installed in thirteen boreholes to record ground water levels prior to start of construction and to determine groundwater fluctuations.

(5) Pump tests were conducted on seven selected boreholes in rock. Borings were pump tested in rock at or near tunnel grade to determine water inflow characteristics during construction.

(6) Piezometers were installed in eight borings. Piezometers were located at multiple levels in rock and overburden to determine the subsurface water characteristics prior to construction and to establish a base for determination of seasonal variation.

(7) Seismic surveys were conducted along the tunnel alignment to determine the depth to rock and to aid in selecting borehole location. Results of the seismic survey are shown on Plates 2 thru 16.

7. INVESTIGATION DURING CONSTRUCTION

a. General. Explorations were made during construction by the contractor for a tail tunnel designed and constructed as a convenience to the contractor for muck removal and for a revised tunnel realignment approved as a cost savings to the Government and the contractor.

(1) The tail tunnel located beneath the Connecticut River east of the outlet shaft was explored using five drive sample borings carried to refusal. The borings did not core rock and were made to assure that there was an adequate rock cover over the tunnel crown which was to be used as a muck car by-pass.

(2) The value engineering tunnel realignment at the intake end of the tunnel had several seismic lines made by the contractor for the purpose of determining the elevation of the rock surface and two borings to tunnel grade to confirm the quality of the rock on the realigned tunnel.

C. GEOLOGY

8. REGIONAL GEOLOGY

The City of Hartford is located in the Connecticut Valley which is a broad generally north-south oriented lowland basin underlain by bedrock of Triassic age consisting of conglomerate, sandstone, and shale with included dikes and sills of basalt. The entire valley up to 108 miles in length and 22 miles in width represents a structural block that is bordered on the east and west by highlands formed of more resistant igneous and metamorphic rocks. The sedimentary rocks of the valley have been faulted and tilted to the east at a dip of 15^{o} to 20^{o}. This physical setting has been modified by glaciation, causing the deposition of till and later lacustrine lake deposits of stratified sands, silts and clays above the rock.

9. SITE GEOLOGY

The relief at the site presents a north-south trend which generally reflects the general strike of the trap rock ridge and of the sedimentary rock which dip gently to the east. The relief is low except where faulting and differential weathering have left prominent ridges of resistant "trap" rock projecting above the floor of the valley. The bedrock is generally blanketed by glacial till which mantles the bedrock surface. In the lowland area, the till is buried beneath extensive lacustrine deposits of stratified sand and varved silt and clay. These deposits formed in glacial lakes grade upward to silts and sands and become integral with the terraces of sands and gravels. The sand and gravel terraces formed in temporary lakes that were controlled by the local spillways during the glacial recession which in the vicinity of the Park River in Hartford approximates

elevation 45 MSL. Recent floodplain deposits of stream sediment and debris occur in local areas along the streams. Depth and composition of these deposits is highly variable. The subsurface water level is controlled by the Connecticut River and other local stream gradients and topography with the upper clay layer creating an impervious boundary that controls subsurface drainage.

10. STRATIGRAPHY

The rocks along the alignment are primarily easterly dipping Triassic sandy shales/siltstones interrupted by a zone of basalt flows and some limited unique rock types near the basalt intrusion. The interpreted stratigraphy based on initial rock core classification and borehole camera interpretation is shown on plates 2 through 16. An exaggerated scale containing the information from the TBM camera and physical sampling during construction is shown superimposed in a graphic legend at tunnel grade on the as-built contract drawings, plates 2 through 16. It should be noted that the camera documentation is broken in areas where functional difficulties occurred in the automatic camera system. The automatic photo documentation is better suited toward identifying structural characteristics of the exposed bedrock rather than identifying specific or general lithology. (For a complete discussion of the camera capabilities, limitations and recommendations see Appendix "A", Final Report, Geotechnical Instrumentation, In-Situ Stress Measurements and Photographic Geologic Documentation, Park River Auxiliary Conduit, Hartford, Connecticut.) Descriptions of the various significant rock types are as follows:

a. Red Shale/Siltstone. The dominant rock type is reddish-brown shale/siltstone. The shale contains sandy phases and is interbedded with gray shales and thin sandstones. It is thin bedded and calcareous. Calcite fills the open bedding planes, joints, and fractures. The shales are usually well-cemented and moderately hard but some zones are classified as soft and weak. The sandy phases are mostly competent and hard to very hard. Bedding strikes roughly north-south and generally dips 10^{o} to 20^{o} to the east but with local variations.

b. Gray-Black Shales. Gray and sometimes black shales are interbedded with the red shales. They are thin-bedded and similarly oriented. The beds are thinner than the red beds and were used as markers to correlate between boreholes. Gray shales are calcareous, moderately hard to soft and are similar in physical properties to the red shales.

c. Sandstone. Thin whitish to gray calcareous sandstone beds are common within the shales. Many sandy zones appear to correlate between boreholes and were used as markers. The beds are hard but sometimes show solution activity and localized concentrated jointing. Variations include a coarse red sandstone (arkose) and a thin zone of interbedded volcanic sandstone and shale that were encountered in boreholes FD-9T and FD-24T respectively and verified in photographs taken by the TBM camera.

d. Basalts. Basalt flows near the intake shaft are oriented consistent with the local stratigraphy although structural modifications are apparent (see plates 2&3). They are usually gray and olive gray (locally black), slightly vesicular and non-vesicular, calcareous, hard and contain healed hairline fractures throughout. Localized broken and weathered zones occur and are noted on the drawings.

e. Aphanite. This gray-fine grained to glassy rock type occurs in borehole FD-9T between the depths of 137 and 188 feet. Its origin is uncertain and it occurs in zones with unresolved structural discontinuities. It is hard to very hard but also contains numerous irregular healed hairline fractures. Some zones may be slightly weathered and less dense. This rock type was not identified by the TBM camera.

11. STRUCTURAL GEOLOGY

a. Jointing. Joints were examined for characteristics in rock core and also logged and oriented by borehole photography. Detail sections on Plates 2 through 16 show significant joint data as interpreted from the TBM camera and geologic logs. The principle joint control applicable to each of the rock types is as follows:

(1) Shales. The primary joints in the shales are along the bedding generally dipping between 10^o and 20^o. Steeply dipping joints are variably distributed as strike and dip joints with irregular to rough surfaces and dips ranging from 70^o to 90^o. Many are very thin and healed with calcite making highly visible mapping by the TBM camera.

(2) Sandstones. Specific joint sets were not identified in the sandstone beds and were considered for design purposes to be random and generally steeply dipping.

(3) Basalt. In borings FD-8T and FD-14T less than 10 percent of the joints were parallel to the trend of bedding in the shale. More than 50 percent strike between N45^{o}E to N45^{o}W and dip 60^o to 90^o. Relatively few joints were visible on the TBM camera.

b. Folds. Folded beds appear present at Sta 57+50. Borehole photography detected a reverse dip of the beds in FD-22T. See plate 8. A fault was considered likely in this zone based on the core samples and its presence was verified by the tunnel construction which intersected a fault at approximately Sta 59+00 and 63+00.

c. Faults were interpreted in two areas, near Sta 57+00 and between Sta 89+50 and 95+00. Fault conditions were experienced during construction between Sta 59+00 and 63+00 based on construction records and between Sta 95+88 and 96+54 based on the TBM camera interpretation.

12. ENGINEERING CHARACTERISTICS OF OVERBURDEN MATERIALS

The stratigraphic sequence of the soils is shown on Plates 2 through 16. This sequence was depicted based on explorations and seismic velocity interpretation. A detailed report of the soils testing and their properties at the shaft locations is contained in DM No. 8 Auxiliary Conduit Shafts. The engineering properties of these materials is as follows:

a. Glacial Till. Materials in the glacial till deposit in the Hartford area are compact, well graded, gravelly silty sands, silty sandy gravels and gravelly sandy silts from fine contents between 25 and 55 percent. The materials range from nonplastic to slightly plastic with liquid limits of less than 25 and plasticity indices of less than 8. Natural water contents are between 8 and 15 percent. Specific gravity ranges from 2.69 to 2.75. Saturated natural unit weights are estimated to be between 140 and 150 pounds per cubic foot. For the anticipated stress ranges these materials are estimated to have shear strengths equivalent to an angle of internal friction of 35 degrees, based on past experience with similar soils. The estimated coefficient of subgrade reaction for these materials is on the order of 400 pounds per square inch per inch.

b. Varved Clay. Reference is made to Design Memorandum No. 5, Embankments and Foundations, Part 1, Box Conduit, for a detailed evaluation of the engineering properties of the materials in the varved clay deposit. The material consists of alternating laminae of soft plastic clays and silts. Liquid limits of the clay bands range from 55 to 75 with plasticity indices berween 25 and 45. The silts have liquid limits varying from 30 to 50 with plasticity indices between 0 and 20. Natural water contents range from 60 to 80 percent for the clays and from 35 to 50 percent for the silts. Specific gravity of the silty bands varies from 2.75 to 2.80 and that of the clay bands from 2.75 to 2.85. Saturated unit weights vary between 100 and 110 pounds per cubic foot. Consolidation test results for this and other projects indicated that the material has been preconsolidated to loads at least 0.5 TSF greater than present overburden stresses. For design purposes the undrained shear strength of the material is estimated to be 500 pounds per square foot.

c. Silty Sands. The materials in the loose to moderately compact zone of silty sands at the outlet shaft are nonplastic, silty medium to fine sand, gravelly silty sand and sandy silt. Saturated natural unit weights are estimated to be between 120 and 135 pounds per square foot. Shear strengths are estimated to be equivalent to an angle of internal friction of 30 degrees.

13. ENGINEERING CHARACTERISTICS OF BEDROCK MATERIALS

A summary of the mechanical rock properties is shown on Table 2. The test results represent physical samples selected from twenty-one explorations on the tunnel alignment, generally distributed over the area of the tunnel section. Detailed results of physical testing is contained in DM No. 9 Auxiliary Conduit Tunnel. General comments on the physical

TABLE 2

SUMMARY OF MECHANICAL ROCK PROPERTIES

		RED SHALE	GRAY SHALE	BASALT	APHANITE	RED SANDSTONE
SPECIFIC GRAVITY (DRY)	TESTS COUNTED	25	4	14	3	2
	RANGE	2.58 - 2.72	2.61 - 2.73	2.68 - 2.87	2.46 -2.62	2.58-2.73
	AVE.	2.66	2.66	2.74	2.54	2.66
UNIT WEIGHT (PCF)	TESTS COUNTED	25	4	14	3	2
	RANGE	161 - 169.7	162.9 - 170.4	167.2 -175.3	153.5-163.5	161 -170.4
	AVE.	166	166	172.2	158.5	165.7
UNCONFINED COMPRESSIVE STRENGTH (PSI)	TESTS COUNTED	19	4	11	3	2
	RANGE	3,242-13,100	4,329-14,740	5,540-13,740	2,700-6,660	9,350-9,536
	AVE.	7,752	8,556	10,263	4,090	9,443
MODULUS OF ELASTICITY E(psi x 10^6)	TESTS COUNTED	7	1	9	1	
	RANGE	.2 - 5	2.5	.89 -10.	3	--
	AVE.	2.1		4.62		

properties of the rock are as follows:

a. Density. The density of the rock averages 2.66 BSSD (166 pcf) for the shale and 2.75 BSSD (171.6 pcf) for the basalt. These rocks comprise the bulk of the rock encountered in the tunnel.

b. Swell Test. Swell tests conducted on the shale rock indicated a lineal expansion normal to the foliation of 0.4 of a percent in approximately 160 hours.

c. Moisture. All physical property tests on the rock were conducted at the natural moisture content. All test samples of shale were waxed in the field to retain their natural moisture and were stripped immediately prior to testing. Absorption averaged 1.18 percent for 27 samples in the shales and sandstones and .566 percent for 15 samples of the basalt.

d. Modulus of Elasticity. Elastic moduli ratios were determined as a by-product of the controlled testing. Design values for the modulus of elasticity was determined from testing to be 2.1 x 10^6 psi for the sedimentary rocks and 4.6 x 10^6 for the basalt. The modulus of elasticity (E) for the sedimentary in a vertical direction were closely related to those obtained by in-situ testing at depths up to 100 feet which average 3.07 x 10^6 psi. Average "E" for greater depths during in-situ testing were appreciably higher being 8.25 and 7.33 x 10^6 psi at depths of 148 feet and 154 feet respectively. In comparison tests performed at tunnel grade during construction averaged 2.96 x 10^6 psi at depths of 183-195 feet. This lower modulus may be attributable to affects of the tunnel excavation method which was done by blasting in this area.

e. In-Situ Stress. The results of in-situ stress at the site prior to tunnel construction are in general agreeable with the regional data having a maximum principal stress with an average orientation of N48° E$\pm$ 12°. There also appears to be relatively low lateral stress with an average maximum principal stress of 452$\pm$ 133 psi compression. The stresses of lower magnitudes occur above the fractured zone between a depth of 104 feet (EL -79) and a depth of 111 feet (EL -86) and slightly higher magnitude occur below the fractured zone. Correlation with the in-situ stress measured at tunnel grade by the overcoring technique indicates an average maximum principal stress in the vertical mode of 291 psi which is slightly less than the lower range of values obtained from the pre-construction in-situ borehole testing.

f. Sliding Friction. Friction tests on natural joint surfaces were conducted on saturated surfaces at 5, 10 and 15 TSF. The average angle of sliding friction for the joint surfaces was $\emptyset$ = 31.6° with an average cohesion of 2.2 TSF primarily due to undulations in the bedding surface. Sliding friction test on a sawed joint in the shale had a $\emptyset$ of 26.1° with a cohesion of 0.59 TSF.

g. Triaxial Tests. Triaxial tests on shale indicate an average angle of internal friction of 48.9^{o} with a cohesion of 188 TSF.

h. Multistage Triaxial Tests. These tests were conducted on steeply dipping natural joints which cross the natural bedding in the sedimentary rocks. This method was used to evaluate the coefficient of friction and cohesion along natural strike, dip and diagonal joints. Due to the irregularity of most joint surfaces the strength values were high resulting in average values for friction of $\emptyset$ = 30.5^{o} and cohesion of 97 TSF.

i. Unconfined Compression Tests. Unconfined compression tests were conducted with and without strain measurements. Three design parameters have been determined from the tests. Compression strengths represents all samples tested. Values varying considerably with the principal rock types are as follows:

ROCK TYPE	AVG. COMP. STRENGTH	AVG. POISSONS RATIO (U)	AVG. MODULUS OF ELASTICITY (E)
Shale	7,892	.29	2.1 x 10^{6} psi
Basalt	10,263	.38	4.6 x 10^{6} psi

j. Dynamic Testing. Selected samples prepared for static testing were sonic tested at varying loads for shear and compression wave velocities. Dynamic properties indicated good correlation between similar rock types and conditions. Compressive wave testing also verified the in place field values obtainedduring the seismic refraction survey.

k. Slaking Tests. Stress relief in the form of sample crazing when immersed in water after air drying were noted in several shale samples during pre-construction testing. Slaking in the mole assembly chamber where drill and blast excavation exposed extensive areas of rock slaking was controlled by immediate application of gunite. Slaking was not observed in the precast portion of the tunnel where contact grouting would serve as a deterent against slaking through exposure.

l. Rock Movement During Construction.

(1) Multiple Position Borehole Extensometers (MPBX). MPBX installed from the ground surface prior to construction were monitored on a daily basis when the tunnel heading was in a position of 100 feet before and 100 feet beyond the instrument location. Readings were continued at a reduced frequency for the completion period of the tunnel. Anchor spacing above the tunnel crown for the ten test sections were

as follows:

Anchor #1	2.0 to 3.7 feet
Anchor #2	3.9 to 5.3 feet
Anchor #3	6.8 to 9.0 feet
Anchor #4	10.9 to 15.7 feet
Anchor #5	19.8 to 26.7 feet
Anchor #6	46.7 to 61.6 feet

Based on the MPBX data, the rock movement measured over the crown of the tunnel excavation was relatively small in areas of competent shale and sandstone rock types. Movement in these areas ranged from .03 to 0.26 inches. The larger of these movements occurred in areas of moderate faulting. The largest readings of crown movement occurred in the area of the greatest degree of faulting at MPBX-10 where movement was measured up to 0.6 inch. Crown movement in the vicinity of the drill and blast section at MPBX-1 were the smallest observed on the project. This deviation from the anticipated may be partially due to the fact that the crown portion of the tunnel was first excavated in an 8 foot wide 16 foot high section and roof bolts installed before removing the lower bench. This method of excavation would allow for a more uniform transfer of the additional stress from the excavation to be distributed into the rock mass by the rock bolts.

The rate of measured rock movement in the sound to moderately sound rock areas appears to be somewhat dependent on the rate of heading advance in the zone 50 to 100 feet past the instrumented test section. At most test section locations practically all of the measured rock movement occurred as the mole passed through the 100 foot zone with most of the movement being measured as the heading advanced through the first fifty feet. While approximately 80% of the measured movement occurred as the heading progressed up to 1.5 months after the heading exited this zone. (See plates 21 through 30 showing rate of advance vs. rock movement.) While the rate of movement seemed somewhat dependent on the rate of heading advance, there was no apparent correlations of the magnitude of rock movement and the rate of tunnel advance. All MPBXs indicated an upward attenuation of the rock movement measured from the tunnel opening.

Two of the MPBXs measured an apparent upward rock movement as the TBM approached the test section. Since the upward movement attenuates toward the ground surface, it is unlikely that movement is caused by a settlement of the reference head. Use of the interactive graphics computer program also indicates an upward movement occurs at the beginning of the velocity release. Review of the printout of the block movement indicates the rotational movement of blocks could cause an upward movement of the keystone block which would produce a negative or upward movement of the lower anchor points. For observation of this effect see plate 25.

14. GEOLOGIC FACTORS AFFECTING TUNNELING

a. General. A composite record in graphic form displays the tunneling rate vs. rock type and structure (see plate 43). The tunneling rate was relatively constant regardless of rock strength once the learning period for operating the mole passed. Tunneling rates were plotted on the basis of feet moled/hour inclusive and exclusive of downtime. Rates are for those portions of the work completed by the TBM exclusive of the drill and blast area where the tunnel boring machine (TBM) was assembled and the tail tunnel constructed to facilitate the removal of muck. The tunnel was advanced up grade from the outlet shaft progressing into the dip of the rock structure which was divided into seven different areas based on the composition and structure of the rock. A summary of the rock structures and the pertinent TBM progress data is as follows:

(1) Area 1. This area was comprised of massive, moderately hard, bedded, red to brown siltstone shale and sandy shale. Length of area is 1,900 feet between stations 25+00 and 44+00. Total moling time was 848 hours. The average rate of advance was 2.5 ft/hr exclusive of downtime for the mole. Total maintenance averaged one hour for every 8.6 feet of advance which included mole maintenance during a cessation of operations for the Christmas holidays. The average moling rate including all delays was 2.2 ft/hr. The wear on cutters resulted in one hour of cutter change for every 22.6 feet of advance in this rock type. Total work hours of 1,068 results in the mole being effectively utilized in drilling for 79% of the operation time.

(2) Area 2. The rock on this section between stations 44+00 and station 57+00 consist of massive, hard, red to black shale with zones of massive sandstone. The 1,300 foot length was moled in 472 hours at an average rate of 2.8 ft/hr. Maintenance on the mole averaged one hour for every 14.1 feet of advance. Changing cutters averaged one hour of cutter change for each 17.1 feet of tunnel advance. This results in the mole being in an effective mole operation 34 percent of the total work time.

(3) Area 3. The rock in the zone between station 57+00 and 63+00 consisted of soft to moderately hard red brown shale containing highly jointed rock in a fault zone. The 600 foot zone was moled in 224 hours at an average of 2.7 ft/hr. Maintenance on the mole averaged one hour for every 5.4 feet of advance with cutter changes at an average of one hour of cutter change for every 18.8 hours of operation. Effective use of the mole in an operational mode was approximately 57 percent of the total work time.

(4) Area 4. The rock in this section between stations 63+00 and 68+00 was a moderately hard, interbedded, moderately thin gray to black sandstone and shale. The 500 feet was moled in a total of 276 work hours with 152 hours spent effectively moling. This results in a progress rate of 3.3 feet per hour with an average of one hour of cutter

change for every 37.3 feet moled and a total maintenance rate of one hour for every 20.8 feet of progress. This results in the effective mole usage for 86% of the total working hours.

(5) Area 5. The rock in area 5 between stations 68+00 and 75+00 consisted of a moderately hard interbedded, thin, red to gray sandstone interbedded with shale. This generally massive rock required 232 hours of moling at a rate of 3.0 feet per hour with one hour of cutter time required for every 43.7 feet of progress with total mole maintenance averaging one hour for every 29.2 feet of progress. Effective operation of the mole was 91% of the total work hours.

(6) Area 6. The rock in this 1300 foot section between stations 75+00 and 88+00 consisted of a hard to moderately hard, red to brown, shale and sandy shale for 700 feet and a black massive jointed basalt for 600 feet. Total hours moling was 425 with 232 hours spent excavating the basalt. This results in a progress rate for area 6 of 2.9 ft. per hour with a rate of 2.6 feet per hour in the basalt only. Mole maintenance was high spending an average of one hour of maintenance for every 7.6 feet of operation and cutter changes for every 19.1 feet of progress. In the basalt only, cutter changes were low requiring an average of one hour of cutter change for every 30 feet of progress; however, total maintenance was high requiring an average of one hour maintenance for every 5.5 feet of progress. Total time effective for mole usage was 72% for the entire zone and 68% for the basalt only.

(7) Area 7. Some of the most difficult rock conditions were encountered in this final section between stations 88+00 and 97+87 at the intake shaft. Rock type was highly variable consisting of a moderately hard to soft and brecciated (soil-like) sandy shale and sandstone. Progress and conditions were affected by the fact that the mole was in a gradual curve which made alignment and control of the mole attitude more difficult. Total moling time required to excavate this section was 432 hours of a total working time of 652 hours. The average mole progress rate was 2.1 feet per hour with an average of one hour of cutter change required for every 11.3 feet moled and one hour total mole maintenance every 4.1 feet. Effective time for mole usage was 66% equal to that of the previous fault zone in Area 3.

b. Summary. The average progress rate for 7,200 feet of tunnel after shakedown of the system averaged 2.71 ft/per hour with mole maintenance averaging one hour for every 11.9 feet of progress. Cutter changes alone averaged one hour for every 24.2 feet of advance. Total operational effectiveness, that is total hours worked/hours moling was 76.5 percent. A summary chart of rock type vs. TBM progress is shown on Table 2. The lowest rateof advance (2.1 ft/hr) and smallest footage per maintenance hours was in Area 7 and the highest rate of advance was

TABLE 3

T.B.M. PROGRESS				
ROCK TYPE	MOLE RATE FT/HR	CUTTER CHANGES FT/HR	TOTAL MOLE MAINTENANCE FT/HR	EFFECTIVE TIME %
SILTSTONE AND SHALE AND SANDY SHALE	2.2	22.6	8.6	79
SHALE WITH MASSIVE SANDSTONE	2.8	17.1	14.1	84
SANDY SHALE MASSIVE	3.3	31.3	20.8	86
SANDSTONE AND SHALE, THINLY BEDDED	3.0	43.7	29.2	91
BASALT AND SHALE	2.9	19.1	7.6	72
BASALT	2.6	30.0	5.5	68
SHALE AND SANDSTONE SOFT, FRACTURED	2.1	11.3	4.1	66
FAULT ZONE	2.7	18.8	5.4	66
AVERAGE	2.7	24.2	11.9	76.5

Area 4. These values are considered to be largely controlled by the rock conditions. The lowest maintenance rate is one hour for every 29.2 feet of advance including cutter changes at the rate of one hour for every 43.7 feet tunneled occurred in Area 5. These low maintenance factors are partially attributed to the uniformity and moderate hardness of the rock in this area.

In summary, the rate of progress depended primarily on rock structure and conditions. Hardness of the rock did not have a predominant effect on the rate of progress; however, the rock structure did decidedly affect the amount of mole maintenance required. Cutter changes were generally less in the massive basalt which had a 30 ft/hour cutter change rate than in the sound interbedded sandstone and shale which averaged 26.7 ft/hr. However, mole maintenance averaged considerably higher with a rate of 5.5 ft/hr for the basalt as compared to 29.2 ft /hr average for the sound interbedded sandstone and shale. This resulted in a mole effective time of 68% for the basalt vs. 91% for the interbedded sandstone and shale.

Review of these results indicate that rock structure i.e. fault zones had a greater effect on mole progress than rock hardness. Rates of advance did not vary greatly between the harder and softer rocks nor did the amount of cutter changes. However, the mole maintenance factor was over 14% higher for the hard rock indicative of high stress on the mole to maintain the same rate of progress.

The effect of geologic factors on tunneling also governed the liner erection which was most difficult in areas of poor rock where roof collapsed time was reduced and filling and grouting of the annular space was more time consuming. Discussion of rock structure on liner loading and erection time is discussed in paragraph 23, Construction Rate of Progress.

In summation, the use of precast liners which were universal for all conditions minimized the effect of geologic conditions on the construction cycle. The rock strength values were within the range which could be handled by the machine at nearly the same rate of progress; however, the progress rate was maintained only at the sacrifice of increased mole maintenance.

15. CONTROL OF WATER

The contractor was required as part of the work to furnish all plant, labor and materials and perform all the work in preventing the waters of the Connecticut River and the Park River from innundating areas necessary for the construction of the tunnel and inlet and outlet structures. The specifications required that the contractor submit for review and approval the control of water scheme, which he intended to implement during construction for each of the project structures.

No separate payment was allowed. All associated costs were included under the applicable contract unit and lump sum prices for the items or structures to which the work pertains.

a. Surface Structures. All surface structures were constructed using a sump pump operating from the excavated area. No dewatering systems external to the excavation were required or utilized. Inflow of water became a significant item only on the outlet shaft where water flows at the rock surface from previously drilled exploration borings created a flow of water which affected construction operations. This condition claimed as a changed condition Mod. No. P00024 (JC#24) was settled by a construction modification at a cost of $18,456.00.

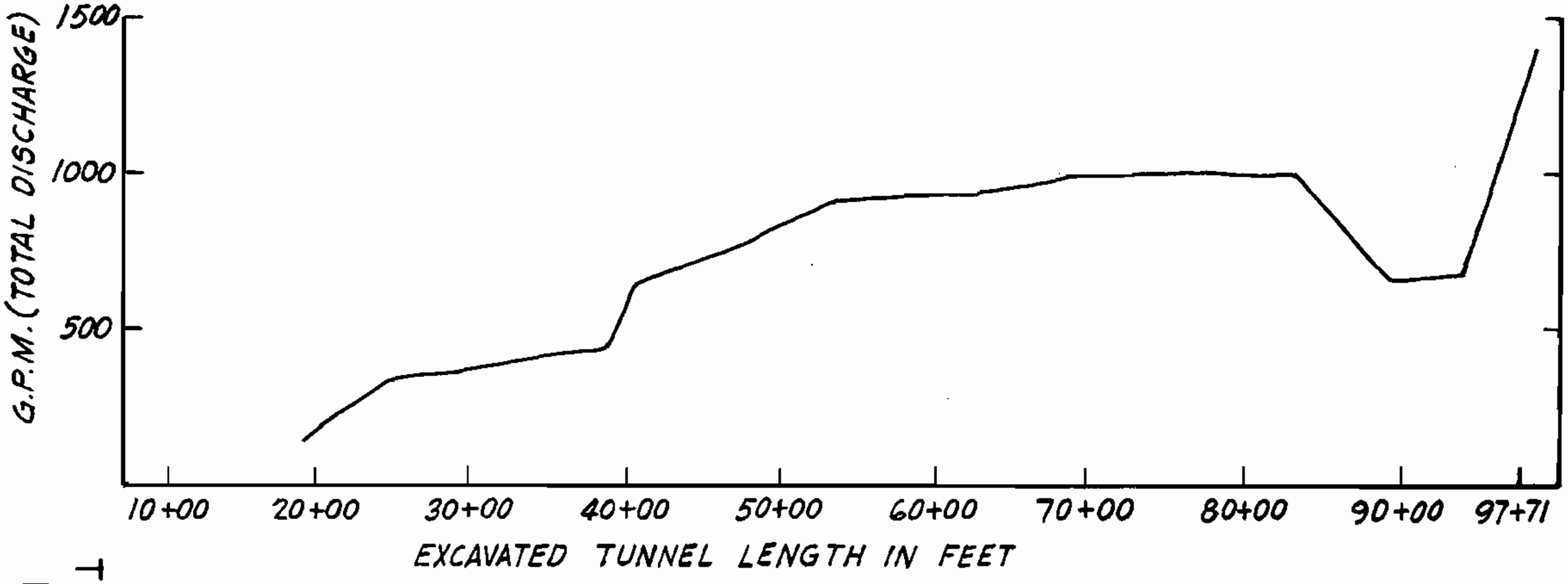
b. Subsurface Structures.

(1) Tunnel. All seepage water was conveyed to sumps at the intake shaft by flow along the completed tunnel invert. The withdrawal of water from the tunnel was required to be controlled in the areas of Stations 7+25 to 32+50 and 92+50 to 98+00 where uncontrolled withdrawal could cause subsidence of above ground structures. The amount of withdrawal was that which would prevent piezometric levels in the glacial till layer from dropping more than 10.0 feet from previously established equilibrium values. For detailed analysis determining the estimated time rate and magnitude of consolidation due to the anticipated changes in groundwater levels and the accepted magnitude of settlement for the structures along the tunnel route, see the report entitled, Report on Effects of Changes in Groundwater Conditions Due to Proposed Construction of Auxiliary Conduit Tunnel by Haley & Aldrich Inc. dated May 1977.

A detailed discussion describing the groundwater fluctuations, the instrument locations, the instruments and the procedures of installation and monitoring are fully described in the Final Report Geotechnical Instrumentation In-Situ Stress Measurements and Photographic Geologic Documentation, submitted by Geotechnical Engineers Inc. dated August 1980 and inclosed as Appendix A. A summary of the piezometer and observation well data is shown on Plate 31 and a record of pump out by total volume and per lineal foot is on Figure 1. Individual water level summaries for each test section are shown on Plates 32 through 39.

Some general observations from the groundwater withdrawal records indicates the following:

(a) The groundwater surface on the eastern end of the tunnel responded to seasonal levels of the Connecticut River. Observation wells at shallow depths were generally unaffected by the tunnel construction and most variations were the resultant of seasonal variations by the Connecticut River.



TUNNEL PUMPING RATES DURING CONSTRUCTION

FIGURE 1

(b) Large drops measured in rock piezometers began when the tunnel heading was within 300 to 500 feet of the piezometer. After the passage of the heading and installation and grouting of the liner, the groundwater level in these piezometers began to recover. The drawdown of the piezometer surface in bedrock is not uniform progressing away from the tunnel to the ground surface. Where a drawdown curve is projected the curve approaches in a cross section that of a pumped well. Piezometers in rock were highly affected by the open joint structure in the rock.

(c) Piezometer levels in the till did exceed the predicted amount but did not cause excessive settlement or damage to the surface structures. Two conditions may be postulated as to the reasons. The first consideration is that due to the thinness of the till layer, the piezometers may have been more sensitive to the sudden drop in the rock water levels. And secondly, as a result the actual pore pressures in the till did not drop to the degree indicated since consolidation of the clay did not generally exceed the maximum settlement considered critical to the structures. A plot of the surface settlement caused by groundwater withdrawal is shown in Appendix B. Maximum settlement at any monument did not exceed ½ inch. Settlement did exceed the recommended ¼ inch minimum at the Church of the Good Shepherd from September 1980 to March 1981 for short periods of time. No damage was observed as a result of these short term maximum settlements in the range of 3/8 of an inch.

Control of water in the specifications was governed by monitoring of piezometers based on a drawdown analysis which would preclude the possibility of surface settlement that could cause damage to structures. Groundwater drawdown due to tunnel construction typically occurred 300 to 500 feet ahead of the tunnel heading indicated by a rapid rate of withdrawal in the rock piezometer with a delayed effect of the piezometers in till (see plate 32). Rates of water discharge with tunnel progress are shown on Figure 1. Unit rates were relatively uniform with a cumulative curve dampened by the rate of sealing of the annular space by pressure grouting. The cumulative effect of the secondary grouting is shown by the reduced outflow in GPM/lf of tunnel as displayed on Figure 1. However, even though the folow was reduced, encountering the fault zone in the vicinity of Station 90+00 caused a rapid rise in the water inflow for a relatively short distance of the tunnel.

16. VIBRATION CONTROL

All vibration control was exercised in the area of conventional excavation for the outlet shaft, inlet shaft and the chamber constructed for the assembly of the mole. The vibration control was exercised by a subcontractor John V. Dinan Associates Inc., Consulting Engineers and Seismologists, Patterson, NJ using a Springnether three component portable seismograph Serial No. 4028-4128. Special requirements limiting peak per component particle velocity had been established in the specifications for particular structures considered susceptible to vibratory effects.

The peak particle velocities established for special structures was as follows:

STRUCTURES & LOCATION	PEAK PER COMPONENT PARTICLE VELOCITY
Church of Good Shepherd 155 Wyllys Street, Hartford, CT	0.7 in/sec
All structures (excluding the Church of Good Shepherd) between Tunnel Stations 7+25 to 33+00 and 92+50 to 98+00	1.2 in/sec
Churches located between Sta 36+00 and 93+00	1.4 in/sec

All other areas were subject to allowable peak particle velocities in accordance with regulatory statutes established by Corps of Engineers Safety Standards and statutes or directives established by State or other authorities. These were generally limited to not exceed an Energy Ratio of one or a peak particle velocity of 1.92 in/sec. A table of recorded values is shown on Table 4. A detailed discussion of the reasons for and procedures used to determine the allowable values limiting the peak particle velocity at special structures is contained in a pre-construction report prepared by Haley and Aldrich Inc. entitled Evaluation of the Effects of Vibrations during Tunnel Construction, Park River Local Protection Project Auxiliary Conduit, Hartford, CT.

In summary, no damage was recorded as a result of vibrations resulting from blasting. Concern for areas of the portals and the mole assembly room was not realized as a result of the contractor's election of Option 3 requiring the use of a TBM (Tunnel Boring Machine). No vibrations were felt at the ground surface as a result of the TBM operation.

TABLE 4

PEAK PARTICLE VELOCITY RECORDED DURING CONSTRUCTION

OF THE PARK RIVER AUXILIARY CONDUIT

Date	Time	Location of Blast	Location of Instrument	Peak Particle Velocity
11/3/78	12:40 pm	Sta 7+65	Colt Building	0.2411
11/7/78	5:25 am	Sta 7+73±	On site	0.1951
11/7/78	7:18 pm	Sta 7+85±	"	0.2049
11/8/78	11:06 am	Sta 7+85±	"	0.5314
11/17/78	3:40 am	Sta 8+67±	"	0.4354
11/22/78	2:37 pm	Sta 9+10±	"	0.3187
11/30/78	3:10 pm	Sta 9+47±	Van Dyke Ave.	0.1307
12/1/78	6:50 am	Sta 9+55±	"	0.1493
12/12/78	4:53 am	Sta 7+13 to 7+40±	On site	0.3394
12/12/78	4:00 pm	Sta 7+52± to 7+67±	"	0.2244
12/12/78	7:40 pm	In tunnel exact location unknown	"	0.0916
12/13/78	12:20 am	"	"	0.2489
12/13/78	5:55 am	"	"	0.1637
12/13/78	10:37 am	"	"	0.0700
12/13/78	3:48 pm	"	"	0.0244
12/14/78	10:25 am	"	"	0.1024
12/14/78	12:20 am	"	"	0.0748
12/14/78	10:55 am	"	"	0.2491
12/21/78	7:30 pm	"	"	0.4886
12/22/78	2:00 am	"	"	0.3588
12/22/78	8:15 am	"	"	0.1979
12/22/78	8:48 pm	"	"	0.3436
1/3/79	6:25 pm	"	"	0.6811

Date	Time	Location of Blast	Location of Instrument	Peak Particle Velocity
1/9/79	1:30 pm	In tunnel exact location unknown	On job site	0.6811
1/9/79	11:15 pm	"	"	0.1095
1/10/79	10:40 pm	"	"	0.1720
1/11/79	8:45 pm	"	"	0.1754
1/12/79	3:55 pm	"	"	0.1469
1/15/79	11:23 am	"	"	0.1574
1/15/79	10:15 pm	"	"	0.1562
1/16/79	2:45 pm	"	"	0.1296
1/16/79	8:10 pm	"	"	0.1907
2/27/80	11:10 am	Intake shaft	50' NW of jobsite	0.0424
2/27/80	12:55 pm	"	50' SE of jobsite	0.1951
2/27/80	2:15 pm	"	100' SW of job site	0.1417
3/3/80	3:40 pm	"	"	1.1986
3/6/80	10:35 am	"	"	0.6782
3/6/80	2:40 pm	"	"	0.4242
4/2/80	10:40 am	"	"	2.0396
4/17/80	12:05 pm	"	-	No data
4/28/80	11:35 pm	"	50' NW of jobsite	0.1723
4/30/80	9:50 am	"	"	0.2345
5/2/80	4:15 pm	"	"	No record
5/6/80	10:10 am	"	"	0.1063
5/6/80	4:00 pm	"	"	0.1979
5/19/80	2:30 pm	"	"	0.1881
5/27/80	1:35 pm	"	50' NW of blast	0.0860
7/7/80	10:05 pm	"	40' W of blast	1.0954

Date	Time	Location of Blast	Location of Instrument	Peak Particle Velocity
7/8/80	8;30 pm	Intake Shaft	50' W of Blast	No record
7/11/80	12:25 pm	Broad St. Shaft	30' E of Blast	"
7/15/80	10:50 am	"	50' SE of Blast	0.2647
7/16/80	9:50 am	"	50' of Blast	0.0670
7/16/80	2:30 pm	"	25' S of Blast	No record
7/17/80	1:15 pm	"	"	0.2032
7/21/80	2:25 pm	"	"	No record
7/22/80	11:25 am	"	"	"
7/22/80	3:30 pm	"	"	"
7/24/80	2:35 pm	"	"	0.0877
7/24/80	4:40 pm	Intake Shaft	Unknown	No record
7/25/80	10:50 am	Broad St. Shaft	25' S of Blast	0.2319
7/25/80	2:40 pm	"	"	0.2898
7/28/80	3:50 pm	"	"	0.3033
7/30/80	10:30 am	"	"	0.2670
7/30/80	3:00 pm	"	"	0.4512
7/31/80	11:10 am	"	"	0.4242
7/31/80	3:15 pm	"	"	0.3181
8/1/80	10:55 am	"	"	0.2963
8/4/80	2:45 am	"	100' SW of Blast	0.0448
8/4/80	2:00 pm	"	25' S of Blast	0.2059
8/5/80	11:00 am	"	"	0.2537
8/5/80	2:15 am	"	"	No record
8/6/80	11:10 am	"	"	0.1969
8/7/80	10:25 am	"	"	0.2109
8/7/80	3:00 pm	"	"	0.1766

Date	Time	Location of Blast	Location of Instrument	Peak Particle Velocity
8/8/80	12:00 pm	Broad St. Shaft	25' S of Blast	0.1766
8/8/80	4:00 pm	"	"	0.1886
8/11/80	12:15 pm	"	"	0.2289
8/12/80	10:20 am	"	"	0.1118
8/12/80	2:15 pm	"	"	0.1039
8/13/80	11:30 am	"	"	0.1232
8/13/80	3:35 pm	"	"	0.1979
8/14/80	3:10 pm	"	"	0.1200
8/28/80	2:00 pm	Location unknown	50' N Intake Shaft	0.1435

D. EXCAVATION PROCEDURES

17. EXCAVATION METHODS

The elected procedure for excavation was the use of a tunnel boring machine (TBM) with a precast liner which was erected beneath the tail shield of the machine. The machine was assembled in a 235 foot long 26 foot high U-shaped chamber upstream of the outlet shaft which was excavated by drilling and blasting using a heading and bench method. Simultaneously a 13.0 foot ± diameter tail tunnel 148 feet in length was driven east of the outlet shaft using a full face excavation operation. The tunnel was supported in the erection chamber by 10 foot long fully encapsulated resin rock bolts at four to five foot centers and shotcrete. The tail tunnel was supported as necessary by random rock bolts without shotcrete.

After completion of the drill and blast section, the TBM was assembled in the excavated chamber and the tunnel advance using the TBM was begun. The TBM was fully shielded, rotary hard-rock machine manufactured by the Robbins Company, Seattle, Washington, which cut a 24 foot 3 inch diameter bore.

18. WATER CONTROL

All control of water was the contractor's responsibility as a part of the project work. The contractor was required to submit for review and approval the control of water scheme which he intended to implement during construction for each of the project features where temporary structures or dewatering methods are required to allow construction in the dry.

Special consideration on withdrawal of water was required in the tunnel between stations 7+25 to 32+50 and 92+50 to 98+00. The purpose of the control was to prevent the observed piezometer levels in the glacial till from dropping more than 10.0 feet as measured from equilibrium values previously established. A plot showing a summary of piezometer observations is shown on Plate 32. Individual water level summaries for each test section 2 through 9 are shown on plates 33 through 40. The reasons for and the analyses of potential settlement effects caused by groundwater withdrawal is contained in a report by Haley & Aldrich, Inc., Cambridge, MA entitled "Report on Changes in Groundwater Conditions due to Proposed Construction, Auxiliary Conduit Tunnel, Park River Local Protection, Hartford, CT" dated May 1977.

The pumping rates and approximate volumes of water pumped during the construction as measured at a control weir at the tunnel outlet is shown on Figure 1.

E. BIDDING PROCEDURES

19. GENERAL

The project was bid based on three different options for tunnel construction and a single shaft construction procedure common to all three options. The use of the option was to obtain the least cost method. Contract specifications required that a contractor could bid on only one option. Fixed prices were established for all line itemsof work required beyond a minimum tunnel design section. Instrumentation was bid lump sum with a Government estimated cost shown on the bid form. Tunnel construction was bid as a line item on a lineal foot basis which included all costs connected with the tunnel construction. During the bidding period all sample test data, design memorandum, rock cores, core photographs and special reports were made available to the contractor, a statement signed by the contractor's representative indicated which reports they had viewed. The contractor was allowed to select rock samples for further testing to determine the physical characteristics of the rock for his bidding purposes.

20. OPTIONS

Three tunnel options were allowed and bid as follows:

a. Option 1. Drill and blast excavation with a variably thick cast-in-place lining. Three sections were designed to meet three ranges of rock loading. All costs of excavating the quantity of rock required for the minimum section were included in the unit price for tunnel construction. The unit price included the entire costs of excavating, transporting, and disposing of excavation materials and maintaining the excavation in a satisfactory condition until the lining was placed.

b. Option 2. Machine excavation including a variable thickness reinforced cast-in-place concrete lining. Three tunnel sections had differing shotcrete thickness and reinforcements to meet three ranges of rock loadings. The unit price bid per foot of tunnel construction included the entire cost of excavating, transporting, and disposing of excavated materials and maintaining the excavation in a satisfactory condition until the concrete was placed.

c. Option 3. Machine excavation with a constant thickness reinforced precast concrete lining. The unit price for tunneling construction included the cost of excavating, transporting, disposing of excavated materials and placement of the precast lining properly installed and back packed with peastone and grouted.

Bid prices for the respective options including construction of the outlet, inlet and Broad Street shafts ranged from $33,374,140 for the drill and blast (Option 1) to $23,248,185 for machine excavation, with a precast lining (Option 3). The greatest number of bids was made on the precast liner option with five of the seven acceptable bids ranging in price from $23,248,185 to $28,551,997.

F. TUNNEL CONSTRUCTION

21. CONSTRUCTION METHOD.

The tunnel was advanced upgrade from the outlet shaft adjacent to the Connecticut River using the Option 3 construction method. Excavation methods are described under Section D, EXCAVATION PROCEDURES. An unlined tail tunnel 13.0 feet diameter by 148 feet long was added by the contractor to allow passage of the muck cars through the outlet shaft area during their unloading (See Plate 16). The tail tunnel was backfilled with tunnel muck and grouted prior to pouring the outlet works lining.

The temporary support and final lining are provided by four segmented precast liner rings which were erected in the tail shield of the TBM approximately 35 to 40 feet behind the cutter face. Each of the four segments is nine inches thick and six feet wide providing a finished inside diameter of 22 feet. Circumferential sponge rubber O-rings were provided between rings and neoprene pad gaskets and a hydraulic cement sealant (Preco-Crete) were used between segments. The invert segment was set on a bed of peastone and the annular space between the excavated rock surface and outside surface of the crown and side segments was filled with 0.5 inch peastone, generally two to three rings behind the last ring installed. Cement grouting of the peastone was generally accomplished in the invert between the second and seventh ring and at the crown and sides from an independent grout gantry about 200 to 500 feet behind the last installed ring. Secondary grouting through the liner was performed in areas of seepage to provide the final sealing of the tunnel liner.

22. EQUIPMENT

Major pieces of equipment used on the construction of the tunnel were as follows:

TBM - Fully shielded, rotary, hard-rock machine, Model Series 240 modified with rotary segment erector, manufactured by the Robbins Company, Seattle, WASH.

Trailing gear with conveyor,exhaust system and muck chute transfer.

Segment and muck cars

Grout Plants (2)

Drills

23. RATE OF PROGRESS (TUNNEL)

The tunnel was started on 11 December 1978 after completion of the outlet shaft. Excavation of the 235 foot long assembly chamber and installation of temporary support which includes rock bolts and shotcrete took approximately 120 days. Assembly of the tunnel boring machine (TBM) began on 3 May 1979 and was ready for operation on 12 July 1979, a total of 142 days. The TBM began operation at station 9+80 on 13 July 1979 and holed through at the intake shaft at station 97+71 a distance of 8,790 feet on 3 July 1980. Total days involved was 355 of which 315 were work days with 210 days of TBM operation. Approximately 85% of the work days were three shift days. Nearly 100% of week end days were one shift days. Of the total project three Saturdays, 33 Sundays and six holidays were not worked.

The average progress of all days was 24.7 feet per day, of working days 28.5 feet per day and for days of mole operations 41.9 feet per day. Weekly progress averaged 172.4 feet per week. Maximum progress per shift was 36 feet and maximum progress per day was 90.0 feet. For a histogram of the rate of progress, see plates 40 through 43.

24. DEVIATIONS FROM DESIGN

Variations from the original Tunnel Abutment occurred in the following areas:

a. Station 10+42 to 42+32, deviations occurred during the start of the mole due to operational difficulties. The boring machine went .595 feet below grade in the first 300 feet. Corrections to a .65% grade from a required .5778% brought the mole back in line in 2,885 feet or station 47+32. The tunnel profile showing the deviations is shown on plates 10 through 16.

b. A value engineering proposal to modify the 200 foot radius curve between stations 95+58.94 and 97+61.70 to a 5,000 foot curve radius was accepted. The change would eliminate the drilling and blasting method of excavation for the last 229 lineal feet of tunneling and allow use of the tunnel boring machine and precast concrete segments to the intake shaft.

c. Deviation occurred from the value engineering proposal alignment between stations 86+17.86 and 97+89.67. Variation from the design alignment varied from the design alignment by two feet below grade and seven feet off the center line at the intake shaft. The deviation from alignment in this section is shown on plate 2.

25. CONTRACT MODIFICATIONS

The contract modifications issued on the tunnel portion of the contract are shown on Table 5, Tunnels and Table 6, Shafts.

26. DESIGN VERIFICATION

a. Tunnel Loads. Rock loads on the precast tunnel liners were estimated during design to range based on the quality of rock structure from 0.26 TSF for the best average and 1.75 TSF for the fault zones. Load computations were based on a density of 2.66 BSSD for the shale and 2.75 BSSD for the basalt. The precast tunnel liner was designed to resist the rock loads as estimated for the worst (fault zone) rock conditions included on Table 2 in DM No. 9, Auxiliary Conduit Tunnel. This value was later reduced 50% in accordance with a conference with OCE on 25 May 1977. The precast liner analyzed in DM No. 9 as an eight segment circular member connected by pins was modified to a four segment unit by VECP #2. (See Table 5, Modification No. P00005.)

The resulting rock loads on the liner are approximated from the stress forces resulting from the strains imposed on the liner (see Table 7). Visual observation indicated that the precast liner had not been overstressed on the final inspection of the tunnel prior to flooding. The magnitude of rock deflections and the resultant strain is shown on plates 21 through 30.

A re-evaluation of the rock loads using the interacting graphic tunnel loading program developed by the Technical Report - GL 79-15 Rational Design of Tunnel Support dated September 1979 is shown on plate 46 as an example of the load configurations by different joint sets.

b. Groundwater. Groundwater was predicted during design based on data from both zone and falling head permeability tests. Rates of water inflow during tunneling predicted in DM No. 9, Auxiliary Conduit Tunnel, Appendix E, that groundwater inflow would be 0.51 gpm/lf for the best average condition and 6.2 gpm/lf for the worst average condition. It was predicted during design that the best average condition would occur in 88% of the tunnel and worst average in fault zones in 12% of the tunnel. Based on this average water inflow in the unlined portions of the tunnel water inflow would approximate 0.6 gpm.

Water inflow records were measured at a V-notched weir at the outlet shaft. Volumes varied based on the volume of ungrouted unsealed tunnel exposed at any particular time. A plot of the water flows records is shown on Figure 1. As is shown the rates vary, however, the mean is approximately 0.5 gpm/lf. The high water inflow in zones projected on the basis of weir measured pump out and the results of secondary consolidation grouting are shown on plates 5 through 19.

TABLE 5

PARK RIVER LOCAL PROTECTION

PROJECT, PART II-AUXILIARY CONDUIT, HARTFORD, CT

CONTRACT NO. DACW 33-77-C-0099

TUNNEL

Description	Reason for Change
Modification No. P00005 (JC #3) The size of the precast segments which make up the tunnel liner is changed from 5' x 9'± to 6' x 18'±	Value Engineering Proposal by contractor (VECP-2) Cost: $17,176.00 Decrease
Modification No. P00007 (JC #7) Provide instrumentation for one ring of the precast test section	To show development of bearing throughout the full length of the tapered longitudinal joint. Cost: $19,082.00
Modification No. P00008 (JC #8) Modify joint seal requirements at transverse and longitudinal joints of the precast concrete tunnel segments.	To get a better seal of the joints based on actual precast segments and loading test performance. Cost: $63,008.00
Modification No. P00011 (JC #12) Secondary grouting of tunnel liners required in areas of high water inflow.	To reduce the amount of water being lost from the various rock formations and minimize the possibility of ground water drawdown through the till layer. Cost: $95,850.00
Modification No. P00012 (JC #13) Change joint sealant for precast tunnel liners from epoxy mortar material to cement mortar material.	1. Evaluation of epoxy material revealed potential for adverse physiological effects on persons working in the tunnel. 2. Surface to receive epoxy sealant must be clean and dry, a condition impracticable to obtain in the tunnel. Cost: $34,984.80

Description	Reason for Change
Modification No. P00016 (JC #17) Provide additional readings for piezometers, observation wells and settlement instrumentation.	Due to delays in completion of the project, piezometer, observation well and settlement readings were required to be read longer. Cost: $19,164.00
Modification No. P00021 (JC#9) Modify alignment of tunnel at west end by increasing radius of curvature to a 5,000 feet radius to allow the tunnel boring machine to excavate and place precast segments to the intake shaft.	Value engineering change proposal by the contractor. (VECP #1) Cost: $81,652.38 decrease
Modification No. P00027 (JC #27) This change which originated as a claim in the VECP #1 area of work provides for payment for additional expenses due to severe fault zone which caused delays and inefficiencies in the tunnel boring operations. In addition, since alignment and grade could not be held, additional work had to be performed at the interface of the tunnel and the intake shaft.	The contractor encountered a fault zone containing clayey material; this condition was claimed to be beyond that which could have been anticipated from plans, specifications, design memoranda and exploration borings performed during the contract period. Cost: $530,196.00

TABLE 6

PARK RIVER LOCAL PROTECTION

PROJECT, PART II-AUXILIARY CONDUIT, HARTFORD, CT

CONTRACT NO. DACW 33-77-C-0099

SHAFTS

Description	Reason for Change
Mod. No. P00016 Provide one foot of crushed stone under outlet structure base slab (Note: This modification includes other items not included in description or cost.)	Stone was needed to provide working base to construct base slab. The material encountered at the elevation required was too soft to support the slab. Cost: $2,550.00
Mod. No. P00017 Buried obstruction	Additional labor and equipment for removal of obstruction Cost: $28,695.00
Mod. No. P00018 (JC #19) Provide payment for Phase II cofferdam sheeting left in place.	Phase II dike had failed on several occasions pointing out possible foundation problems. Leaving sheeting in place increased the stability of the foundation area and tended to prevent damage to outlet structure slab. Cost: $40,000.00
Mod. No. P00019 (JC #20) This change which originated as a claim required a more expensive earth support system for the intakeshaft than that which the contractor submitted for approval.	The original contract drawings provided a loading diagram for the contractor to base his earth support design on. This diagram did not include the loads from Pope Park. Cost: $57,706.00
Mod. No. P00020 (JC #21) Additional rock excavation of Broad Street drainage shaft. (Note: This modification includes other work for which description and cost is not given.)	Rock was encountered at Elev. +38 rather than at Elev. +38 as shown on the drawings. Cost: $ 2,580.00

Description	Reason for Change
Mod. No. P00024 (JC #24) This change which originated as a claim provided for controlling water in outlet shaft by consolidation grouting.	Contractor claimed a changed condition for artesian flow from the rock at the outlet shaft. A review of the claim showed that the water was coming from boreholes put down by the Government during site surveys. Cost: $18,456.00
Mod. No. P00028 (JC #28) This change which originated as a claim provides for changes in the configuration and construction procedures for Phase I and Phase II cofferdams at the outlet structure area.	Fill material from excavation did not have sufficient stability to hold the proposed dikes as called for on the original plan. Cost: $120,000.00

TABLE 7

TUNNEL LOADS						
TEST SECTION NO.	PRE-CONSTRUCTION			POST-CONSTRUCTION		
	ROCK LOAD OPTION 1 TSF	ROCK LOAD OPTIONS 2 & 3 TSF	LINER * OPTION 3 TSF	ROCK LOAD MPBX - CRT TSF**	LINER STRAIN OPTION 3 TSF	OVERLOAD + UNDERLOAD - TSF
1 (Sta 8+20)	.52	.26	.14		CIP Liner	
2 (Sta 11+32)	.52	.26	.14	0.162		
3 (Sta 15+25)	.52	.26	.14	0.112		
4 (Sta 24+00)	1.10	.70	.35	0.049	(WORK IN PROGRESS)	
5 (Sta 27+00)	1.10	.70	.35	0.031		
6 (Sta 43+50)	.52	.26	.14	0.031		
7 (Sta 58+55)	.52	.26	.14	0.118		
8 (Sta 61+19)	.52	.26	.14	0.031		
9 (Sta 91+00)	.52	.26	.14	0.124		
10 (Sta 95+54)	2.4	1.75	0.875	0.307		

*Liner Design Loads Reduced per Conf. OCE on 5/27/77

**Rock Load by Interactive Graphic Method

G. SHAFT CONSTRUCTION

27. INTAKE STRUCTURE.

a. Construction Method. Earth excavation was made with temporary support by a single row of PZ-27 sheet piles supported by internal W 24x100 whalers at approximately ten foot intervals from elevation 40.0 feet to the top of rock at approximately elevation 0.0 as shown on plate 47. Additional support was provided by internal framing with diagonal corner support comprised of W 14x119 whalers. The intake box foundation was excavated in rock from elevation 0.0 to -16.29 in two lifts using 17 to 18 foot deep pre-split holes for the periphery of the cut. Pre-split holes were 18" CC except corners which were reduced to 6" CC.

Shaft excavation in rock consisted of a transition section from the base of the intake box at elevation -16.29 to -41.07 using a pre-split corner round to a circular section of a 26.5 foot diameter. The shaft was then excavated using a 25.5 foot circular section using peripheral pre-split holes 1.5 foot on centers from elevation -96.57.

All blasting was performed using a Hercules Unigel with a powder factor for the 3/4 inch blast holes of 1.35 VF.

b. Equipment. Excavation removal performance in both the earth and rock was by a 100 ton crane and bucket. Material was handled with the assistance of a TD450 dozer, a 977 loader and a Bobcat tractor.

c. Rate of Progress. Excavation inside of the shaft began on 26 October 1979 at approximately elevation 45 and was completed to the top of rock on 21 February 1980. The first blast of the pre-split holes was made on 27 February 1980 and was completed with the drilling and blasting of the tunnel and shaft construction on 27 September 1980. Concreting of the shaft began on 28 September 1980 and was completed to elevation 40 on 16 April 1981. Details of the shaft construction progress are shown on Plate 44.

d. Deviations from Design. No major deviations from design occurred at the shaft excavation, however, the shaft was redesigned to accommodate the tunnel misalignment which occurred in the area of VE #1 after the mole drifted off course in an area of fractured rock. This required a revision of the lower portion of the shaft which is shown in Plate 47.

e. Contract Modifications. Two modifications were involved at the intake shaft. The first was for a change resulting from a modification to the load diagram provided on the contract drawings and by the tunnel misalignment in relationship to the shaft. The initial modification is shown on Table 6 Modification No. P00019 and the second modification No. P00027 on Table 5.

f. Design Verification. Support systems were monitored by use of cross section surveys during construction. No strain gages of other monitoring devices other than the audible failures system were installed. As built drawings showing the final modified shape of the structure is shown on Plate 47.

28. OUTLET SHAFT

a. Construction Method. The shaft was constructed by driving PDA and PZ27 sheeting from the existing ground surface at elevation +13 to elevation -36.0 whereas the original scheme was to continue the excavation with liner plates to the top of rock at elevation -70. Settlement of the upper sheeting which was bottomed in an extension clay zone required a change of procedure and sheeting was driven inside the upper section from elevation -33 to bedrock following the installation of approximately five rings of five gage liner plates. Inside whalers varied from W 6x25 upper to WF 12x65 at the lower elevations. A concrete ring was poured between the upper and lower sheeting and liner plates were used to extend the upper sheeting to produce a cofferdam to elevation +27. Spacing of the shaft rings varied from four to three feet. Excavation was made in the earth section by use of a clam shell and dozer.

Rock excavation in the shaft was performed in half sections using 55 to 60 shot holes at depths of 6 to 12 feet in the initial cut followed by perimeter trim shots at a four foot depth on 12 inch centers. Explosives used were Hercules Unigel on the cut shots with Hercules E-Cord and Hercosplit on the perimeter trim shots. Approach to tunnel grade was made with bell out shots which made the transition to the horseshoe shaped mole fabrication chamber. The chamber was constructed in two lifts separated into upper and lower section at El -131±. Peripheral holes in the tunnel section used full pilot rounds and presplit holes on slash rounds.

b. Equipment. Excavation removed in both earth and rock was performed by a 100 ton crane and bucket. Material was handled with the assistance of a 977 loader and Bobcat tractor.

c. Rate of Progress. Excavation within the shaft area began on 1 February 1978 and reached rock at elevation -72 on 5 August 1978. Rock excavation proceeded immediately and was completed to elevation -150 on 11 December 1978 with the cleanup and concrete invert being poured on 30 January 1979. No activity took place at the outlet shaft during the driving of the tunnel and work was reinitiated on 16 September 1980 with the fabrication of the cofferdam. Work was completed on August 21, 1981 with topsoil and seeding. (See plate 45)

d. Deviation from Design. Several deviations from the planned construction procedures occurred as are discussed in paragraph a. Construction Methods. Further deviations occurred as result of the change in procedures with a resulting offset in the vertical alignment

by one foot six inches to the northwest. The final as built structure is shown in detail on Plate 48.

e. Contract Modifications. Five contract modifications amounting to $209,701 occurred in the area of the outlet shaft. The first modification No. P00016 occurred as a part of a modification which included additional piezometers, observation wells and settlement readings and also contained among other items the cost of placing one foot of crushed stone under the outlet structure base slab. The second modification No. P00017 was for labor and equipment to remove a buried concrete slab and wood piles encountered during shaft excavation. The third and fifth change orders which were the largest incurred were for changes incurred due to the failure of dikes. See Table 6, Modification Nos. P00018 and P00028. The fourth modification, Mod. No. P00024 was a changed condition for water encountered during the excavation in rock. A more detailed account of the description of the modifications and the reason for change is shown on Table 6.

f. Design Verification. Design verification was provided by electrical instrumentation of the ring beams using two strain gages each on the web and the flanges. Measurements were made until failure occurred in some of the gages. Monitoring continued in the measurements of the beam deflection by physical surveys. A safety system with an audible failure warning was also installed to warn construction personnel of any unusual deformations in the structure.

29. BROAD STREET SHAFT

a. Construction Method. The shaft was excavated in earth using liner plates and hand excavation methods. Excavation was made in rock using a guide pilot hole and drill and blast methods. The lining was a grouted cast-in=place RCP pipe.

b. Equipment. Primary equipment used for construction was a clam shell and hand labor.

c. Rate of Progress. Excavation for the shaft was performed in the earth portion by the excavation and installation of liner plates for the first 29.0 feet. Rate of excavation including liner plates was 2.6 feet per day. Excavation in rock by a drill, blast and muck sequence for the next 121.0 feet was at the rate of 4.8 feet per day.

d. Deviation from Design. No deviations from design was made other than a change in the section in rock from Elev +28.0± to Elev 38.0± to tunnel crown at Elev. 84.259.

e. Contract Modification. A single contract modification No. P00020 was issued for $2,580.00 which included the change of rock elevation as noted under paragraph d. and other work for which description and cost is not given.

f. Design Verification. No design verification was performed on the Broad Street Shaft other than the completed intersection with the tunnel crown which was located within a few inches of the center line of the tunnel crown.

H. POSSIBLE FUTURE PROBLEMS

30. CONDITIONS THAT COULD PRODUCE PROBLEMS

Conditions encountered during tunneling between Stations 95+42 and 96+75 resulted in high strains in the crown of the precast liner section and failure of the invert at the test section at Station 95+54. The high strain accompanied by increased loads from the hydrostatic head of water during dewatering of the tunnel will result in liner stresses in excess of those encountered during construction and subsequent filling of the tunnel.

31. RECOMMENDED OBSERVATIONS

During any future dewatering it is recommended that close observation be made of MBX No. 10, anchor No. 1 (Elevation -71.0) located at the test section at Station 95+54. This anchor deflected 0.6 inches during installation of the tunnel and any further downward movement could represent a failure of the liner. If this should occur, dewatering should be done with caution as the liner may require internal support to prevent progressive failure of the liner and possible daylighting of the tunnel crown to the area of Park Street and Pope Park.

I. INSTRUMENTATION

32. A complete documentation of the instrumentation including the plan, type, uses and installation details is contained in the report entitled Final Report, Geotechnical Instrumentations, In-Situ Stress Measurements and Photographic Geologic Documentation Park River Auxiliary Conduit, Hartford, CT. A copy of this report is attached as Appendix C to the as-built foundation report.

J. DESIGN VERIFICATION

33. SUPPORT ASSUMPTIONS

The tunnel support assumptions recommended in DM No. 9, Auxiliary Conduit Tunnel, Site Geology, Foundations, Concrete Materials and Detailed Design of Structures, dated December 1976 were modified prior to issuance of the plans and specifications.

The liner design levels which were predicted on the rock load configuration contained on Table 2 of DM No. 9 were modified at a conference with OCE representatives on 24 May 1977 to one half the previously recommended liner design loads. The rationale for this reduction was based on an assumed reduction of liner required support based on allowance in EM 1110-2-2901 as follows:

(1) Use of steel sets as temporary support in the drill and blast with an RQD of 30 or less becomes a permanent part of the finished lining.

(2) Use of permanent resin bolts as temporary support would reduce liner support requirements.

(3) In all mole operations the ultimate loading is reduced due to the minimal shattering effect on the rock by the mole coring operation. Final liner design loadings were for RQDs of 80, 40 and 30. Loads were respectively 0.275 TSF, 0.55 TSF and 1.2 TSF for drill and blast and 0.14 TSF, 0.35 TSF and 0.875 TSF for mole operation.

Support assumptions for all other portions of the work are as shown in DM No. 9.

34. GROUND CONDITIONS

The subsurface conditions discovered during construction did not appear to exceed the predicted support assumptions. The measured rate and magnitude of loading as measured at the test sections is shown on Plates 21 through 30. A reworking of the geologic information obtained during construction into the computerized interactive load indicated that the magnitude and direction of loading did not exceed the assumed design conditions. A comparison of stresses measured in the liner segments and their relation to the magnitude of deflection in the rock measured at the respective test section is shown on Table 7.

K. CONSTRUCTION HISTORY SUMMARY

35. The notice to proceed on Contract DACW 33-77-C-0099 was issued to Roger Au on 27 September 1977 and based on the contractor's construction progress chart (see Plate 44) submitted for approval on October 3, 1977 had a contract completion date (revised) of 26 September 1980. The actual construction was completed at the outlet works on 1 September 1981 with the final topsoil and seeding. In summary, the outlet works activities opened and closed the work effort on the project since all major activities were carried on through this shaft. Major changes in schedules occurred due to a five month delay during excavation of the outlet works and a four month delay in arrival and assembly of the mole. The machine moled and lined the tunnel in twelve months as opposed to a nine month projected schedule. A detailed profile of the rates of progress for each

of the structures and a discussion of deviations from the contractor's pre-construction progress scheduled for the major features of the contract is shown on Plates 44 and 45 and are in Sections F and G. The principal milestone of the construction from the notice to proceed on 27 September 1977 was the start of construction at the outlet shaft on 1 February 1978. Earth excavations at the outlet shaft was completed on 20 September 1978 and rock excavation was completed to tunnel grade on 11 December 1978. The TBM assembly chamber and the back tunnel was completed by 3 May 1979 and the mole was assembled and became operational by 13 July 1979. The TBM holed through at the intake shaft on 3 July 1980 having traveled a distance of 8,730 feet.

The intake structure excavation in earth started on 6 October 1979 reaching the rock surface on 24 January 1980. Rock excavation was completed to grade on 4 June 1980, one month in advance of the TBM reaching the intake tunnel.

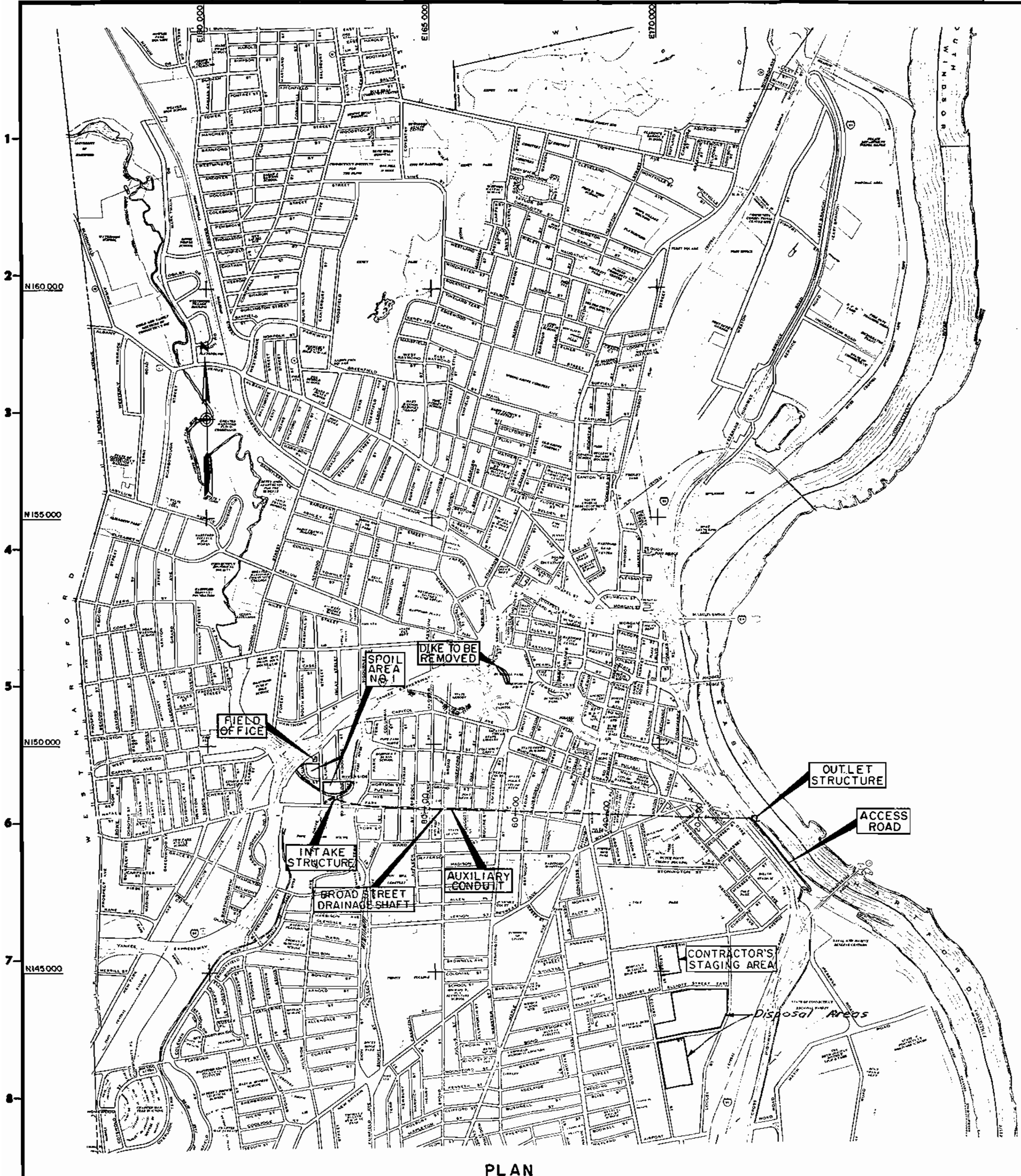
The tunnel became operational on 2 July 1981 after final lining of the intake and outlet shafts and final payment was made on 25 May 1982.

Final cost was $24,457,493 including all adjustments and modifications, an increase of 5¼% over the contractor bid price of $23,248,185 and a decrease of 1% below the Government estimate of $24,729,815.

L. OPERATIONAL HISTORY

39. Since completion of the project, the highest flow passed by the tunnel was 3,400 cfs or 63% of the design capacity in June 1982. The tunnel is now operated by the Greater Hartford Flood Commission who report that they have no difficulties with the operation of the auxiliary tunnel. All available instrument data including surface monuments and MPBXs are continuing to be read by the Greater Hartford Flood Commission as a part of the operational agreement.

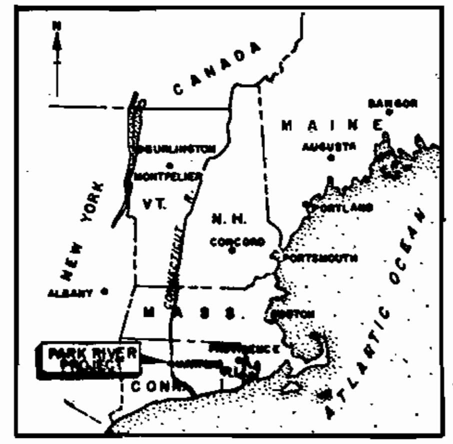
CORPS OF ENGINEERS | U. S. ARMY



PLAN
SCALE: 1" = 1000'

LEGEND

NEW	EXISTING	DESCRIPTION
×365.0	365.0	Elevations
x—F—x	x—EF—x	Fence
		Tree line
		Grade to drain
		Stone wall
		Limit of contractors work area
		Earth cut and rock cut
		Earth fill
		Earth fill with rock slope protection
		Earth surface (Section)
		Assumed bedrock surface or rock cut (Section)
● FD-5		U. S. Army Corps of Engineers boring
S-9		Seismic Line
		Tree
		Deciduous tree
		Evergreen tree
		Shrub
—NSD—	—SD—	Storm Drain
■ DMH	□ DMH	Drain Manhole
■ DI		Drain Inlet
—NSS—	—SS—	Sanitary Sewer
● CB	○ CB	Catch Basin
● SMH	○ SMH	Sanitary Manhole
● NW	○ EW	Water Line
		Gate Valve
●	○	Pole
	—G—	Gas line
	●	Foundation Test Boring N x Size
		Foundation Test Boring 4" x 5½"
		Bedrock Exposures



LOCATION MAP
SCALE IN MILES
40 0 40 80

SPECIAL CONSTRUCTION NOTES
ON
AS-BUILT CONTRACT DRAWINGS

INTERPRETIVE GRAPHIC LEGEND OF ROCK TYPE FROM TBM CAMERA

- Red to red-brown, massive bedding structure; probably siltstone or shale
- Interbedded structure, generally thin to moderately thin beds; probably sandstone interbedded with shale or siltstone
- Gray to black, massive bedding structure; probably siltstone or shale, possibly basalt
- Calcite stringers along bedding
- Jointing approximately normal to bedding, calcite-healed in most cases, generally the joints truncate at contacts between differing lithological units
- Somewhat random jointing pattern, in most cases calcite-healed; probably basalt
- Fault zone, in some cases fines present on brecciated surfaces, in others, calcite-healed.

ROLL 23 ROLL 22 / 94 — Camera Roll No. and Intermediate Sta. No.

AREAS OF SECONDARY GROUTING

23 bags — volume of cement take and special notes on water inflow

TEST SECTIONS

MPBX-1 — Multi Position Borehole Extensometer of Test Section

WATER LEVEL MEASUREMENTS

PZ-6 — Piezometer Location
OW-1 — Observation Well

GENERAL NOTES:

1. Elevations refer to Mean Sea Level Datum.
2. All elevations are plus except as noted otherwise
3. Plane coordinates refer to Metropolitan District coordinate system.
4. Figures in hexagons indicate item numbers under which payment will be made.
5. Bid item references are shown on the drawings for convenience only. Under no circumstances should these items be used without reference to the specifications which describe the work and bid item. Should there be any discrepancies between the work covered by the bid item numbers as shown on the drawings and in the specification, the specifications shall govern.

VALUE ENGINEERING PAYS DIVIDENDS

SAFETY PAYS

GRAPHIC SCALE
1" = 1000' 1000 0 1000 2000

77 0099

REVISION	DATE	DESCRIPTION	BY
		Final field corrections	

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

DES. BY R C | DR. BY J M | CK. BY W.J.R.

SUBMITTED: CHIEF, LAYOUT SECTION
APPROVAL RECOMMENDED: CHIEF, TECH. ENG. BRANCH
REVIEWED: PROJECT ENGINEER
APPROVAL RECOMMENDED: CHIEF PROJECT BRANCH

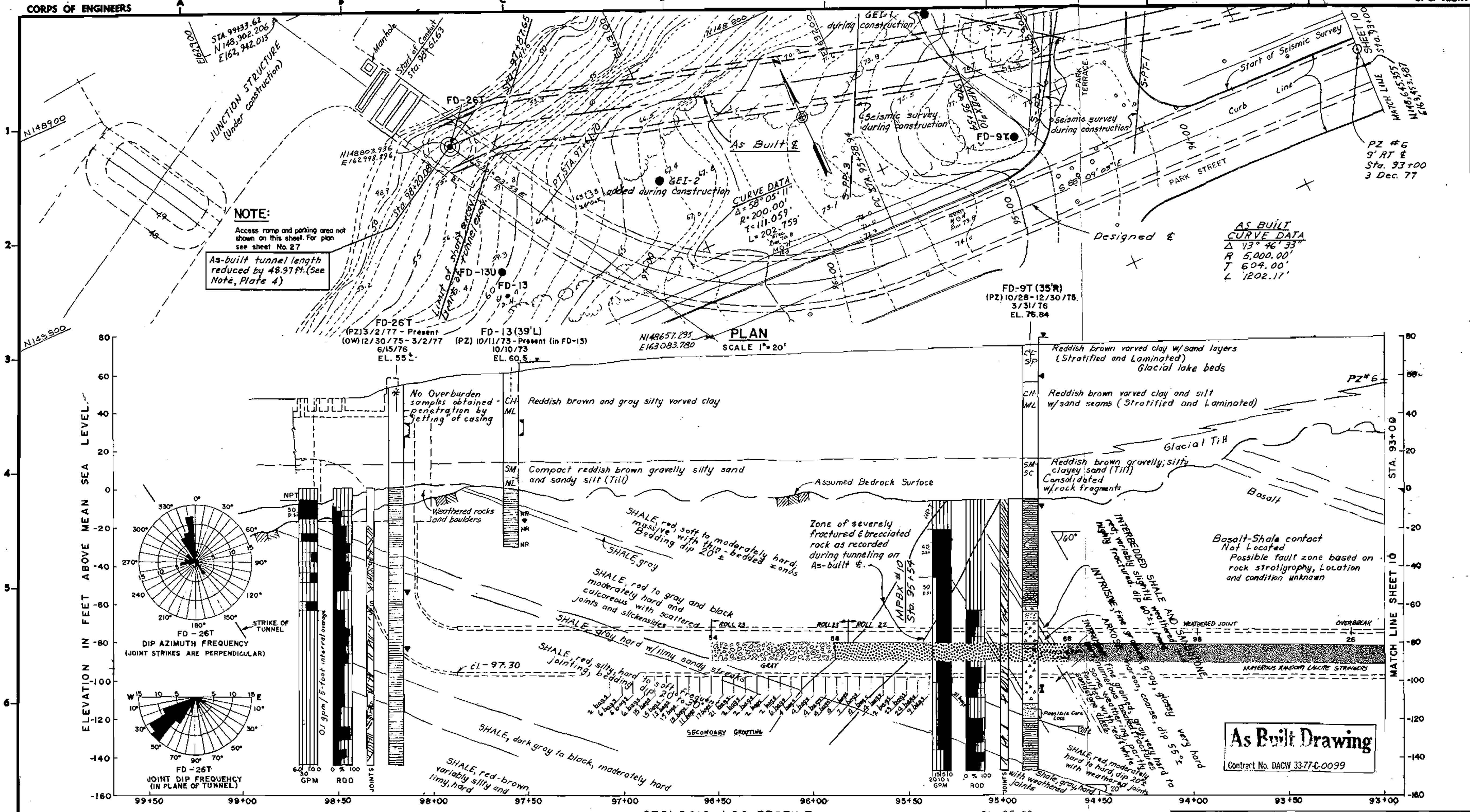
WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT
GENERAL PLAN
HARTFORD CONNECTICUT

APPROVED: for CHIEF, ENGINEERING DIVISION
DATE: JUNE 1977

SCALE: AS SHOWN | SPEC. NO. DACW33-77-B-0023

DRAWING NUMBER: PAR-2
SHEET 2

PLATE I



SCALE OF HARDNESS

VERY HARD	CANNOT BE SCRATCHED WITH KNIFE
HARD	DIFFICULT TO SCRATCH WITH KNIFE
MODERATELY HARD	EASILY SCRATCHED WITH KNIFE; CANNOT BE SCRATCHED WITH FINGERNAIL
SOFT	CAN BE SCRATCHED WITH FINGERNAIL
VERY SOFT OR PLASTIC	CAN BE INDENTED EASILY WITH THUMB

LEGEND

- SANDSTONE, CALCAREOUS
- BASALT AND INTRUSIVE FLOW
- INTERBEDDED SANDSTONE AND SHALE
- BROKEN ZONE AND/OR BRECCIATED
- SHALE, SILTY
- S-"P" SEISMIC LINES

INFLATED ZONE- AN UPPER ZONE OF ALTERATION IN THE ROCK WHICH IS LESS DENSE DUE TO FRACTURING & CHEMICAL DECOMPOSITION

LEGEND FOR GRAPHIC LOGS

FD-20T — TYPE AND NUMBER OF EXPLORATION
(OW/PZ) 1-7/15/74 — OBSERVATION WELL/PIEZOMETER AND PERIOD OF OBSERVATION
1/7/73 — MONTH/DAY/YEAR EXPLORATION COMPLETED
EL. 31± — ELEVATION OF GROUND SURFACE

- MAXIMUM ARTESIAN HEAD
- SUBSURFACE WATER LEVEL
- ARTESIAN FLOW ENCOUNTERED
- RANGE OF SUBSURFACE WATER LEVEL
- BOTTOM OF PIEZOMETER
- UNIFIED SOIL CLASSIFICATION SYSTEM SYMBOL
- NR/NS NO RECOVERY/ NOT SAMPLED
- ROCK TYPE SYMBOL

"A" GPM "B" RQD "C" JOINTS

NOTES:

A. NPT-NO PRESSURE TEST PERFORMED ASTERISK DENOTES SECTION COULD NOT BE SEALED FOR TESTING 50 P.S.I.-CONSTANTLY MAINTAINED PRESSURE FOR 1 TO 5 MINUTES. VOLUME LOSS IN GALLONS PER MINUTE UNDER CONSTANT PRESSURE. TESTS CONTINUOUS AS SHOWN ON LEGEND. SCALE EXPANDED AS INDICATED FOR CLARIFICATION OF LOW FLOW LOSSES.

B. ROCK QUALITY DESIGNATION - PERCENT OF CORE PER RUN WITH PIECES LONGER THAN 4 INCHES.

C. DIP AND FREQUENCY OF JOINTS. "S" INDICATES SLICKEN SIDES ON NATURAL JOINT SURFACE.

NOTES CONT.

D. FOR AVAILABILITY OF DETAILED ROCK LOGS, COLOR PHOTOGRAPHS OF ROCK CORE, ETC. SEE SP-3: PHYSICAL DATA OF SPECIFICATIONS

GRAPHIC SCALE 1" = 20'

VALUE ENGINEERING PAYS DIVIDENDS

SAFETY PAYS

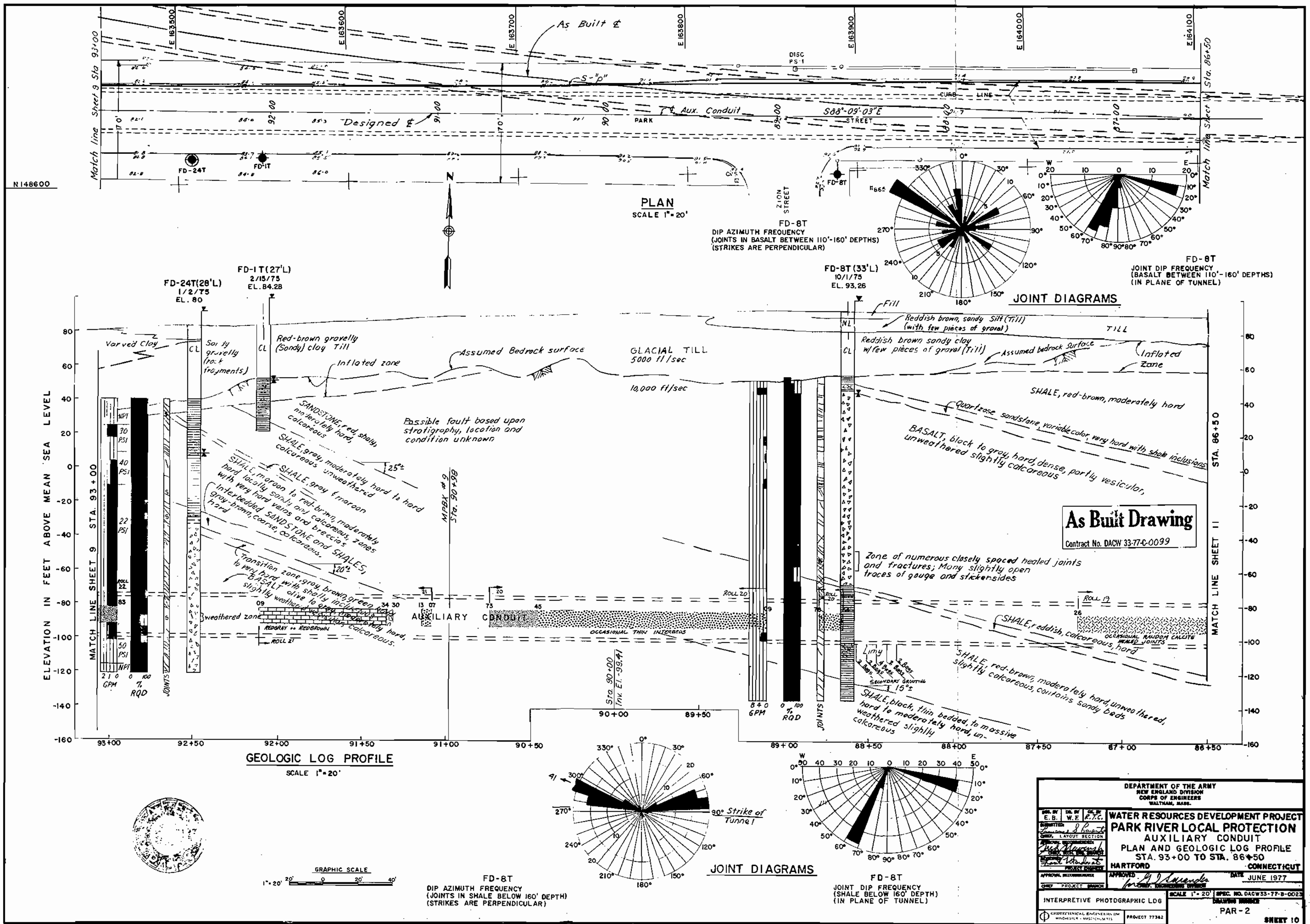
REVISION	DATE	DESCRIPTION	BY
		Final field corrections	

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT
PLAN AND GEOLOGIC LOG PROFILE
STA. 99+00 TO STA. 93+00
HARTFORD CONNECTICUT
DATE JUNE 1977
INTERPRETIVE PHOTOGRAPHIC LOG
SCALE 1" = 20'
SPEC. NO. DACW33-77-B-0023
PAR-2
SHEET 9

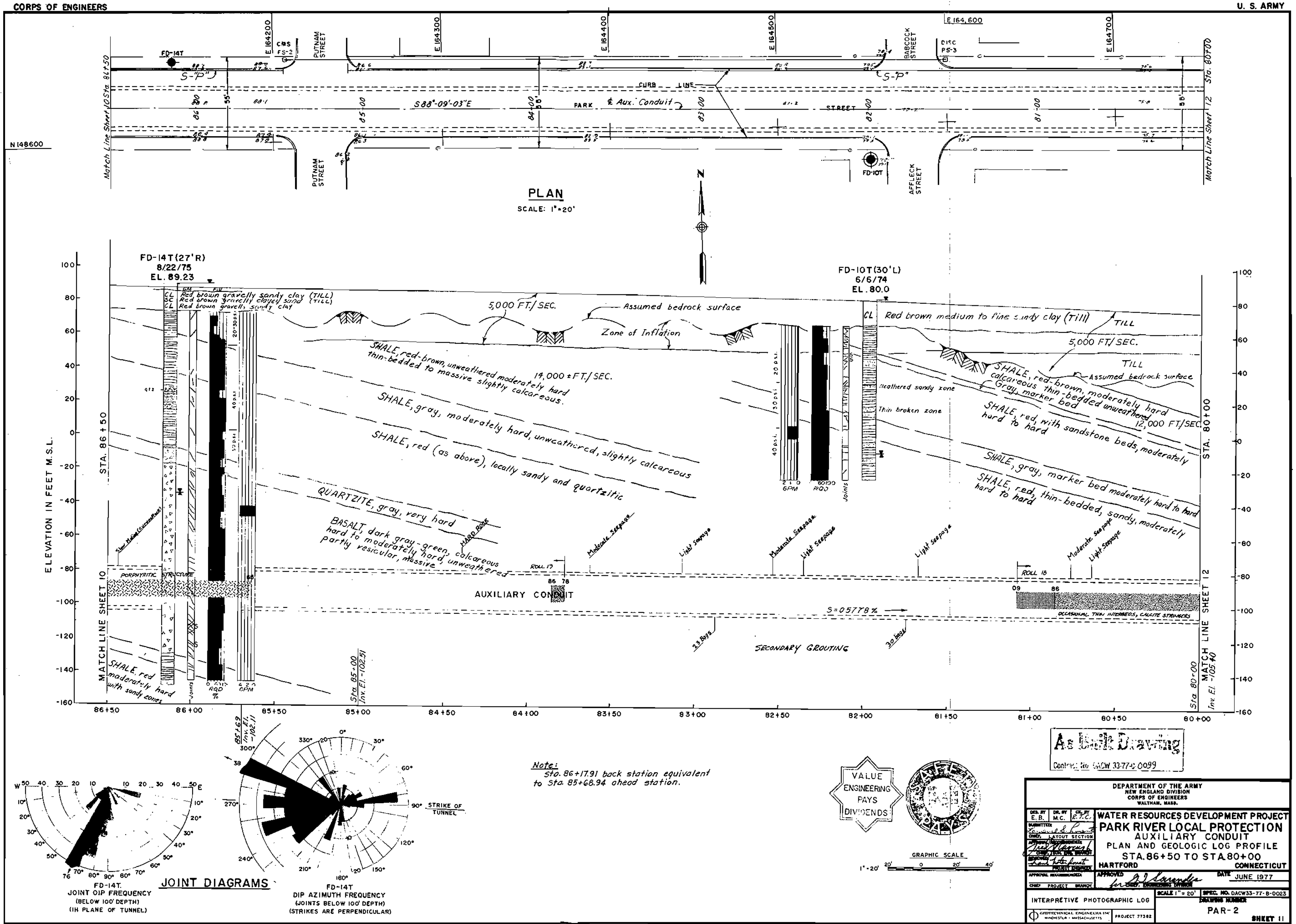
FOR GRAPHIC LEGEND PLEASE REFER TO PLATE OF THIS REPORT

PLATE 2



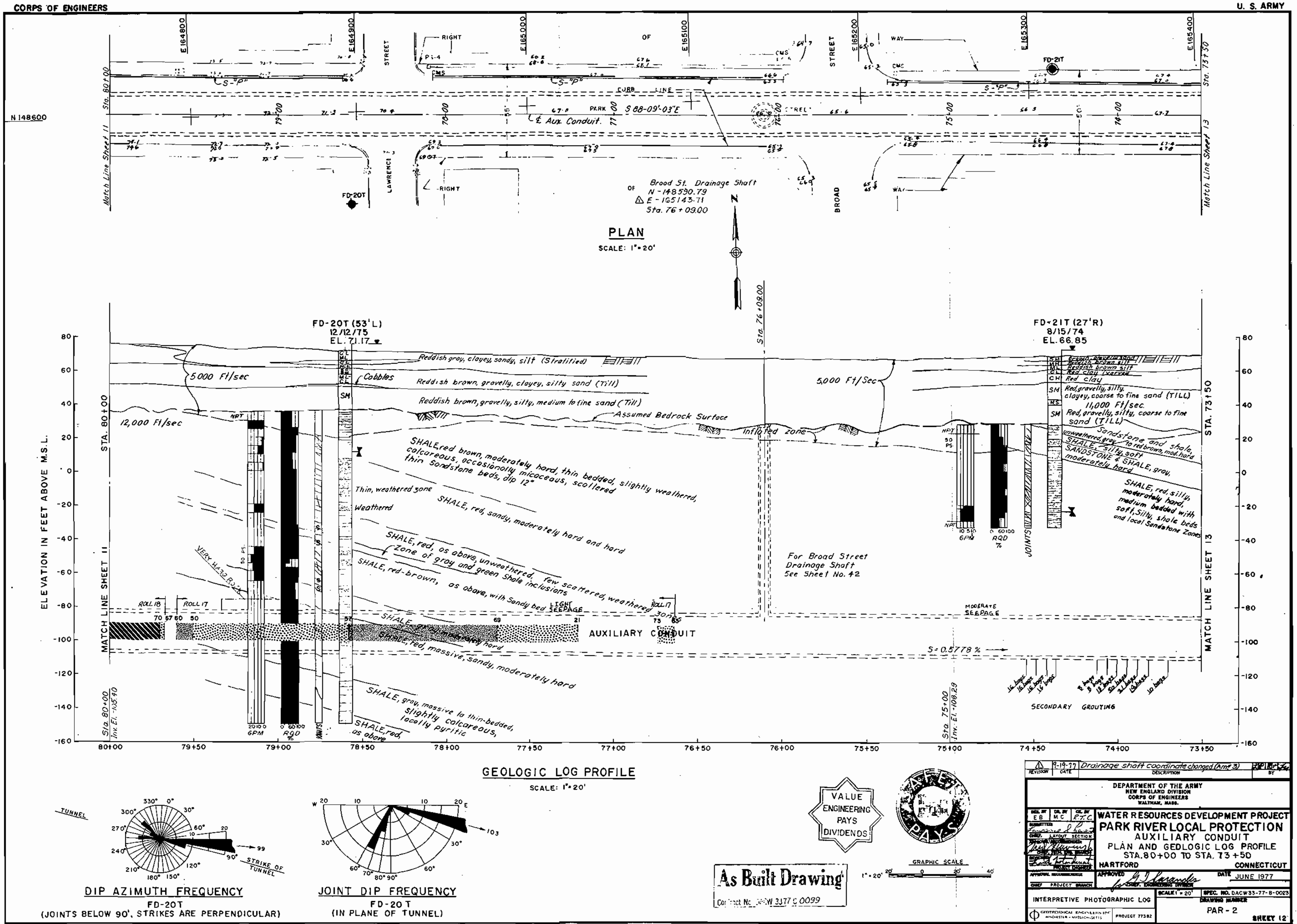
PLAN
SCALE 1"=20'
As Built ℄
Designed ℄
℄ Aux. Conduit
S88°-09'-03"E
PARK STREET
CURB LINE
ZION STREET
Match line Sheet 9 Sta. 93+00
Match line Sheet 11 Sta. 86+50
N 148600
E 163500
E 163600
E 163700
E 163800
E 163900
E 164000
E 164100
DISC PS-1
FD-24T
FD-1T
FD-8T
FD-8T
DIP AZIMUTH FREQUENCY
(JOINTS IN BASALT BETWEEN 110'-160' DEPTHS)
(STRIKES ARE PERPENDICULAR)
FD-8T
JOINT DIP FREQUENCY
(BASALT BETWEEN 110'-160' DEPTHS)
(IN PLANE OF TUNNEL)
JOINT DIAGRAMS
FD-24T(28'L)
1/2/75
EL. 80
FD-1T(27'L)
2/15/75
EL. 84.28
FD-8T(33'L)
10/1/75
EL. 93.26
ELEVATION IN FEET ABOVE MEAN SEA LEVEL
MATCH LINE SHEET 9 STA. 93+00
MATCH LINE SHEET 11 STA. 86+50
Varved Clay
Silty gravelly (rock fragments)
Red-brown gravelly (Sandy) clay Till
Inflated zone
Assumed Bedrock surface
GLACIAL TILL
5000 ft/sec
10,000 ft/sec
Fill
Reddish brown, sandy Silt (Till) (with few pieces of gravel)
TILL
Reddish brown sandy clay w/few pieces of gravel (Till)
Assumed bedrock surface
Inflated Zone
SHALE, red-brown, moderately hard
Quartzose sandstone, variable color, very hard with shale inclusions
BASALT, black to gray, hard, dense, partly vesicular, unweathered slightly calcareous
SANDSTONE, red, shaly, moderately hard, calcareous
SHALE, gray, moderately hard to hard calcareous unweathered
SHALE, gray & maroon
SHALE, maroon to red-brown, moderately hard locally sandy and calcareous, zones with very hard veins and breccias
Interbedded SANDSTONE and SHALES, gray-brown, coarse, calcareous, hard
Transition zone, gray, brown, green, very hard with shaly inclusions
BASALT, olive to gray, slightly weathered, calcareous
Possible fault based upon stratigraphy, location and condition unknown
MPBX #9 Sta. 90+98
weathered zone
RED GRAY to RED BROWN
ROLL 21
AUXILIARY CONDUIT
OCCASIONAL THIN INTERBEDS
Sta. 90+00 Inv. El. -99.41
Zone of numerous closely spaced healed joints and fractures; Many slightly open traces of gouge and slickensides
SHALE, reddish, calcareous, hard
OCCASIONAL RANDOM CALCITE HEALED JOINTS
SHALE, red-brown, moderately hard, unweathered, slightly calcareous, contains sandy beds
SHALE, black, thin bedded, to massive hard to moderately hard, unweathered slightly calcareous
Limy
SECONDARY GROUTING
GPM
% RQD
JOINTS
NPI
PSI
GEOLOGIC LOG PROFILE
SCALE 1"=20'
93+00
92+50
92+00
91+50
91+00
90+50
90+00
89+50
89+00
88+50
88+00
87+50
87+00
86+50
As Built Drawing
Contract No. DACW 33-77-C-0099
GRAPHIC SCALE
1"=20'
Strike of Tunnel
FD-8T
DIP AZIMUTH FREQUENCY
(JOINTS IN SHALE BELOW 160' DEPTH)
(STRIKES ARE PERPENDICULAR)
JOINT DIAGRAMS
FD-8T
JOINT DIP FREQUENCY
(SHALE BELOW 160' DEPTH)
(IN PLANE OF TUNNEL)
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT
PLAN AND GEOLOGIC LOG PROFILE
STA. 93+00 TO STA. 86+50
HARTFORD
CONNECTICUT
JUNE 1977
SCALE 1"=20'
SPEC. NO. DACW33-77-B-0023
INTERPRETIVE PHOTOGRAPHIC LOG
PROJECT 77382
PAR-2
SHEET 10
FOR GRAPHIC LEGEND PLEASE REFER TO R.L.I. OF THIS REPORT
PLATE 3

CORPS OF ENGINEERS U. S. ARMY

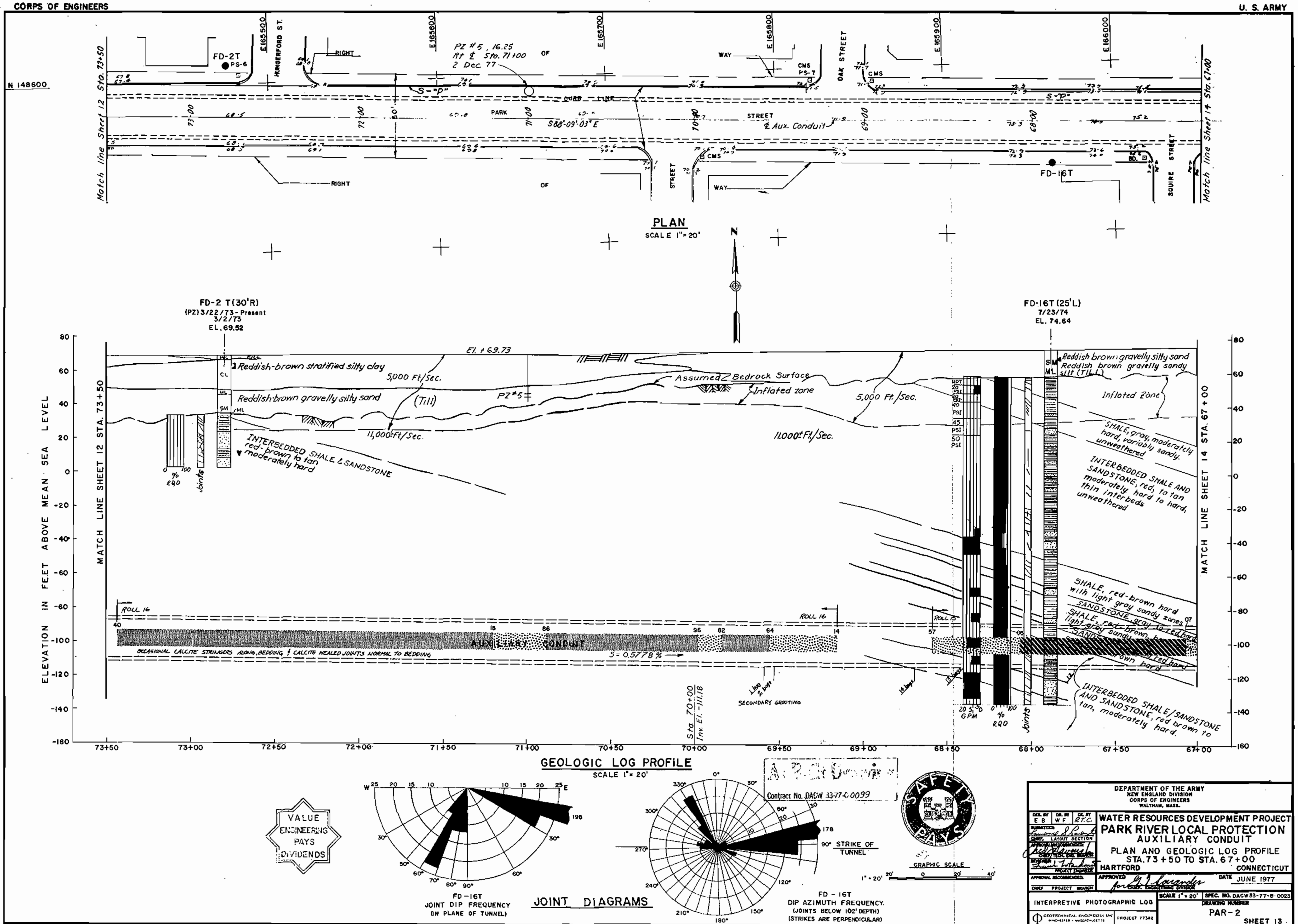


FOR GRAPHIC LEGEND PLEASE REFER TO PL.1 OF THIS REPORT

PLATE 4

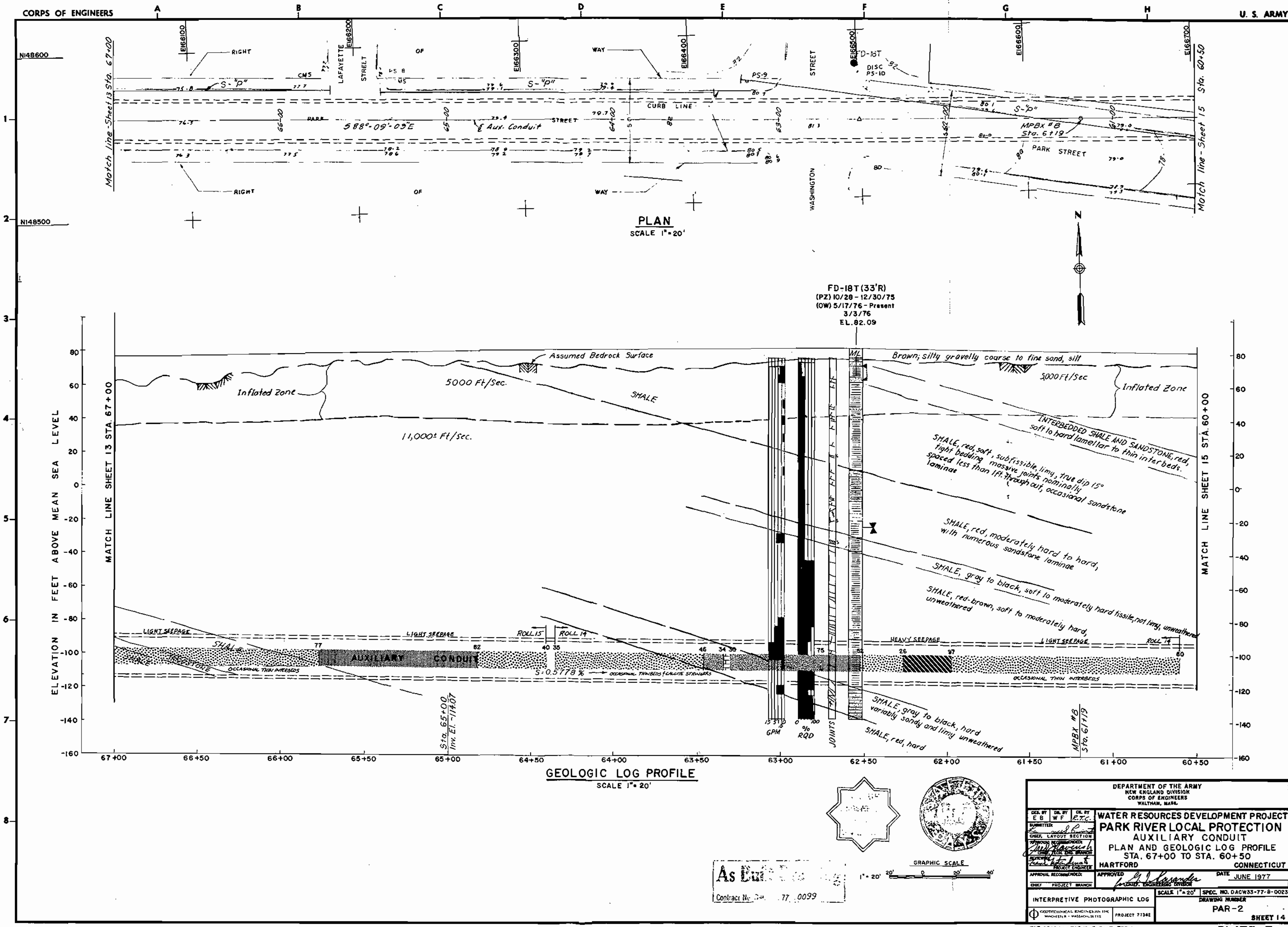


FOR GRAPHIC LEGEND PLEASE REFER TO RLI OF THIS REPORT



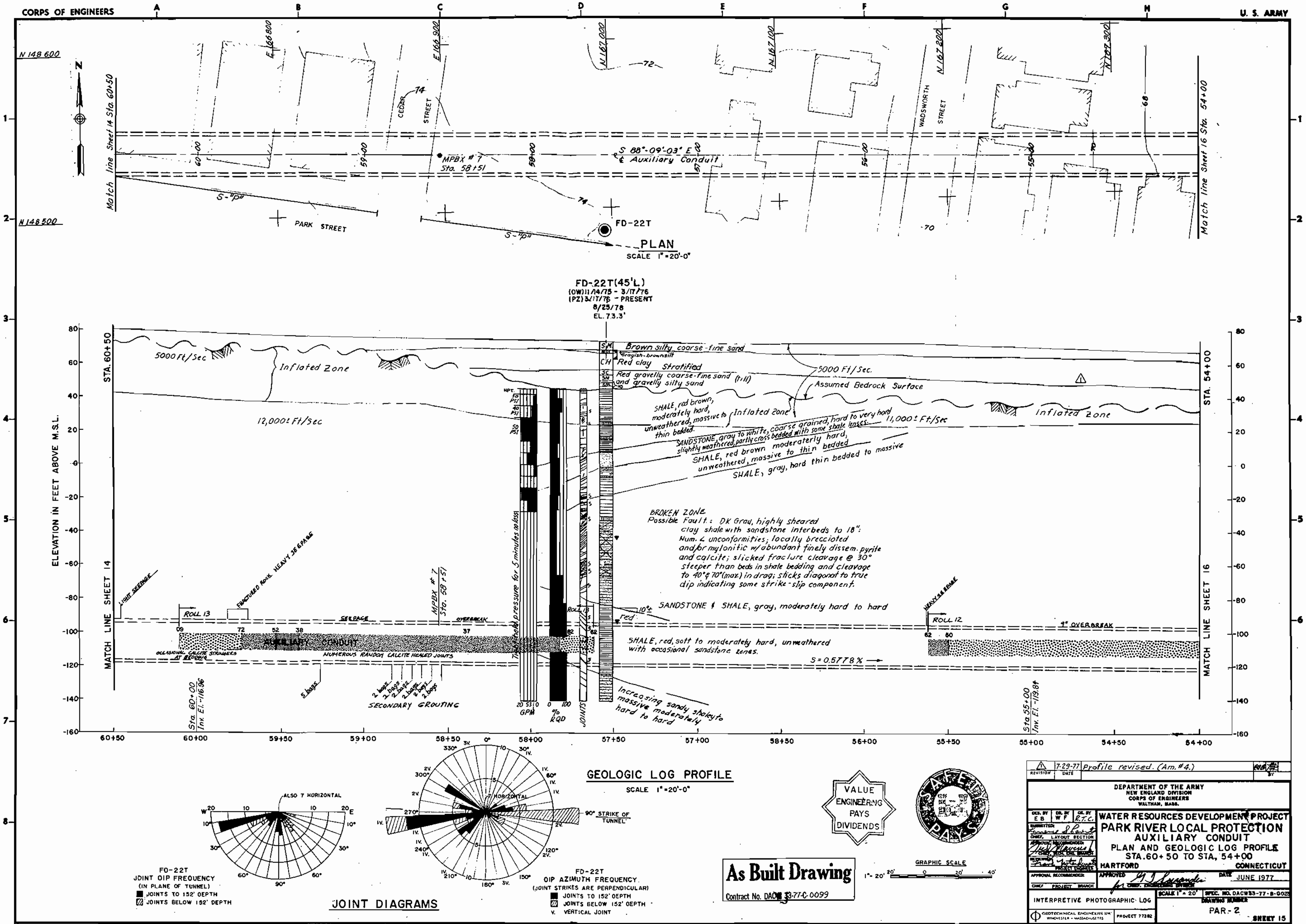
FOR GRAPHIC LEGEND PLEASE REFER TO ALI OF THIS REPORT

PLATE 6



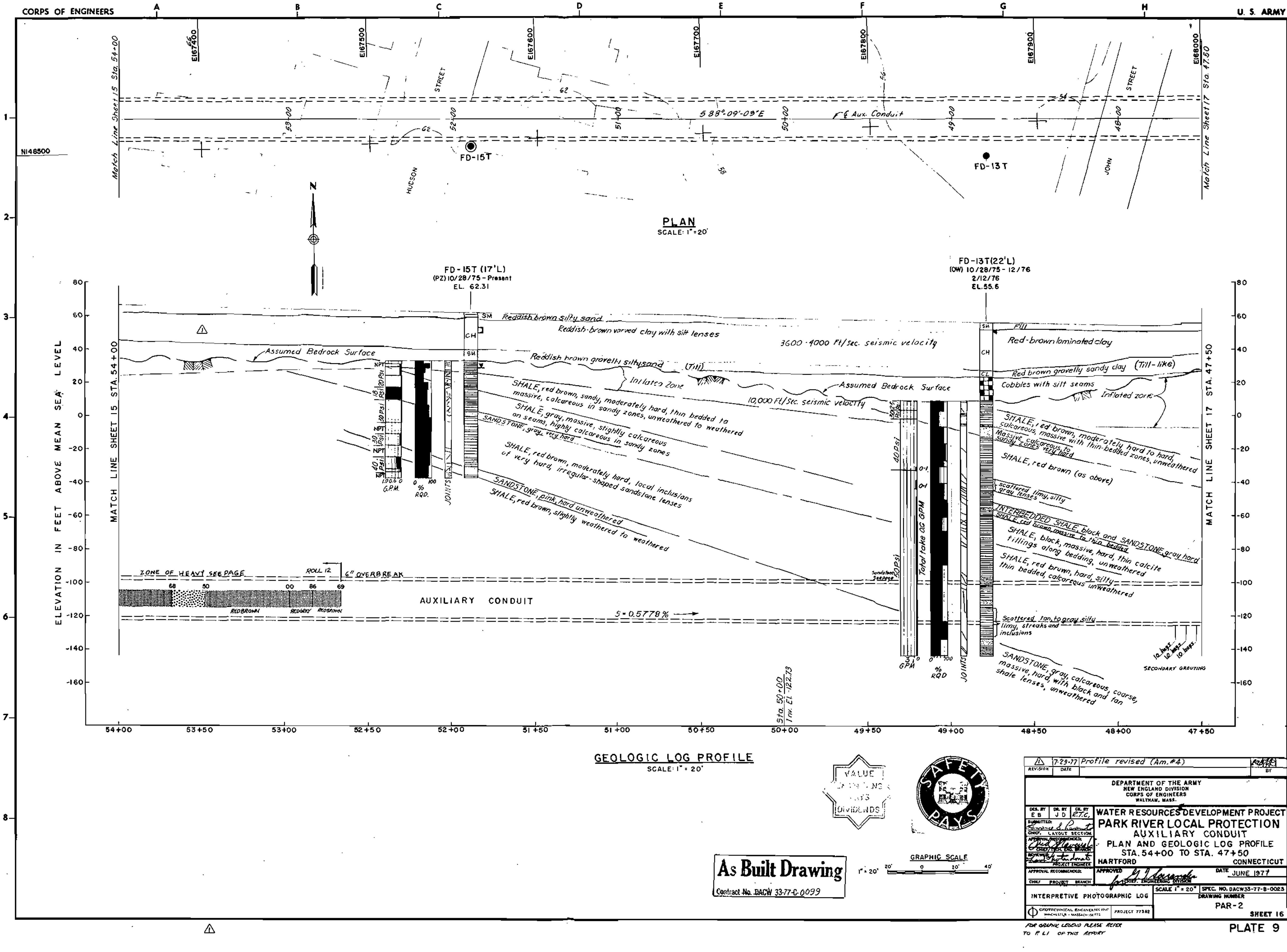
FOR GRAPHIC LEGEND PLEASE REFER TO RLI OF THIS REPORT.

PLATE 7



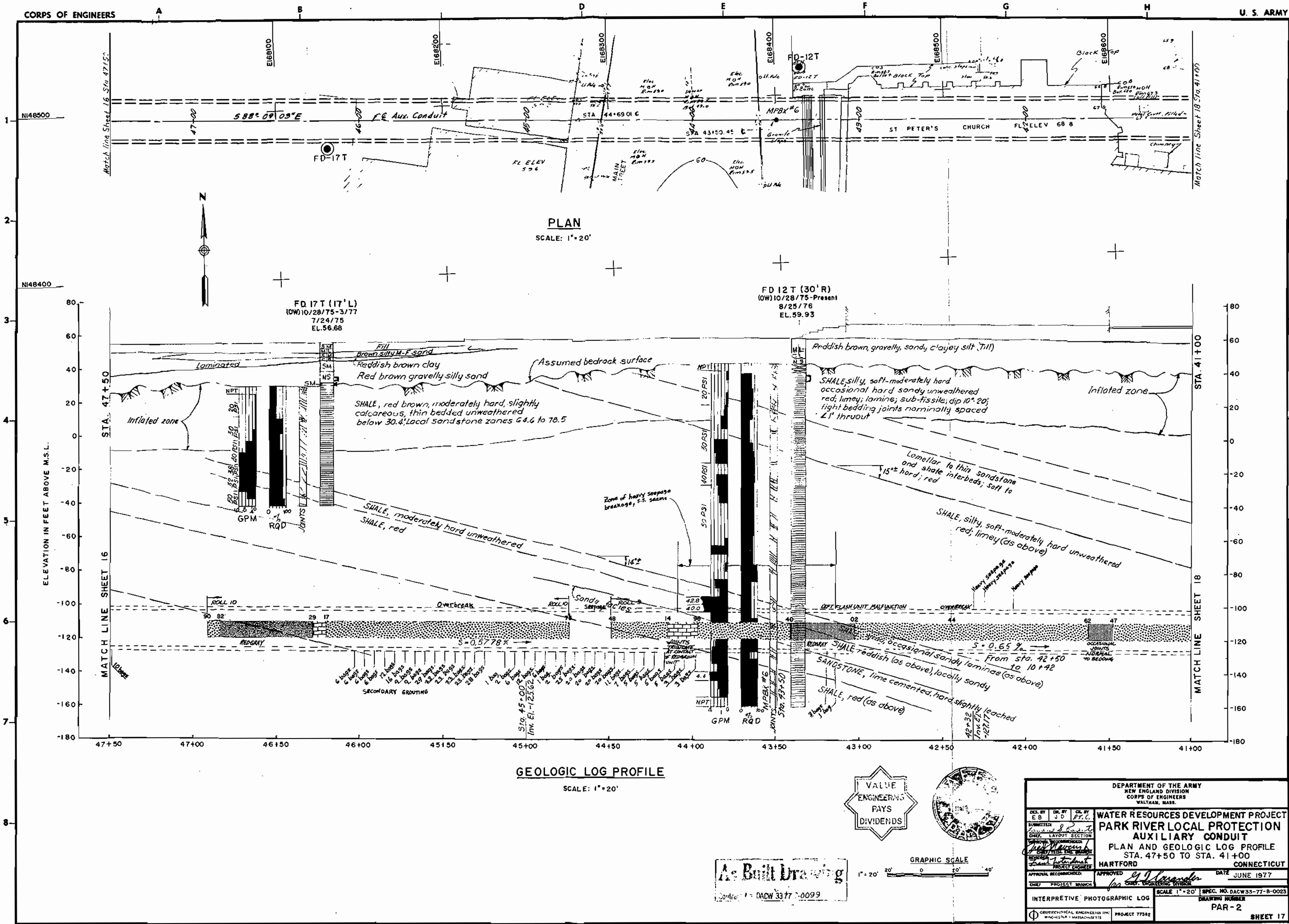
FOR GRAPHIC LEGEND PLEASE REFER TO P.II OF THIS REPORT.

PLATE 8



CORPS OF ENGINEERS
U. S. ARMY
PLAN
SCALE: 1" = 20'
Match Line Sheet 15 Sta. 54+00
Match Line Sheet 17 Sta. 47+50
N148500
E167400
E167500
E167600
E167700
E167800
E167900
E168000
HUDSON STREET
JOHN STREET
S 88°-09'-09" E
℄ Aux. Conduit
FD-15T
FD-13T
FD-15T (17'L)
(PZ) 10/28/75 - Present
EL. 62.31
FD-13T (22'L)
(OW) 10/28/75 - 12/76
2/12/76
EL. 55.6
ELEVATION IN FEET ABOVE MEAN SEA LEVEL
MATCH LINE SHEET 15 STA. 54+00
MATCH LINE SHEET 17 STA. 47+50
Reddish brown silty sand
Reddish brown varved clay with silt lenses
3600-4000 Ft/sec. seismic velocity
Assumed Bedrock Surface
Reddish brown gravelly silty sand (Till)
Inflated Zone
10,000 Ft/sec. seismic velocity
SHALE, red brown, sandy, moderately hard, thin bedded to massive, calcareous in sandy zones, unweathered to weathered
SHALE, gray, massive, slightly calcareous on seams, highly calcareous in sandy zones
SANDSTONE, gray, very hard
SHALE, red brown, moderately hard, local inclusions of very hard, irregular-shaped sandstone lenses
SANDSTONE, pink, hard unweathered
SHALE, red brown, slightly weathered to weathered
Fill
Red-brown laminated clay
Red brown gravelly sandy clay (Till-like)
Cobbles with silt seams
Inflated zone
SHALE, red brown, moderately hard to hard, calcareous, massive with thin-bedded zones, unweathered
Massive, calcareous to sandy zones very hard
SHALE, red brown (as above)
scattered limy, silty gray lenses
INTERBEDDED SHALE, black and SANDSTONE gray hard
SHALE, red brown massive to thin bedded
SHALE, black, massive, hard, thin calcite fillings along bedding, unweathered
SHALE, red brown, hard, silty thin bedded, calcareous unweathered
Scattered tan to gray silty limy streaks and inclusions
SANDSTONE, gray, calcareous, coarse, massive, hard with black and tan shale lenses, unweathered
ZONE OF HEAVY SEEPAGE
ROLL 12
6" OVERBREAK
REDBROWN
REDGRAY
REDBROWN
AUXILIARY CONDUIT
S = 0.5778%
Sta. 50+00
Inv. El. -122.73
Total take 0.6 GPM
Sandstone Seepage
G.P.M.
% RQD.
JOINTS
10 bags
12 bags
10 bags
SECONDARY GROUTING
54+00
53+50
53+00
52+50
52+00
51+50
51+00
50+50
50+00
49+50
49+00
48+50
48+00
47+50
GEOLOGIC LOG PROFILE
SCALE: 1" = 20'
VALUE ENGINEERING PAYS DIVIDENDS
SAFETY PAYS
As Built Drawing
Contract No. DACW 33-77-C-0099
GRAPHIC SCALE
1" = 20'
7-29-77 Profile revised (Am. #4)
REVISION DATE BY
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT
PLAN AND GEOLOGIC LOG PROFILE
STA. 54+00 TO STA. 47+50
HARTFORD
CONNECTICUT
DATE JUNE 1977
SCALE 1" = 20'
SPEC. NO. DACW33-77-B-0023
DRAWING NUMBER
PAR-2
SHEET 16
INTERPRETIVE PHOTOGRAPHIC LOG
GEOTECHNICAL ENGINEERS INC.
PROJECT 77382
FOR GRAPHIC LEGEND PLEASE REFER TO PL. 1 OF THIS REPORT

PLATE 9

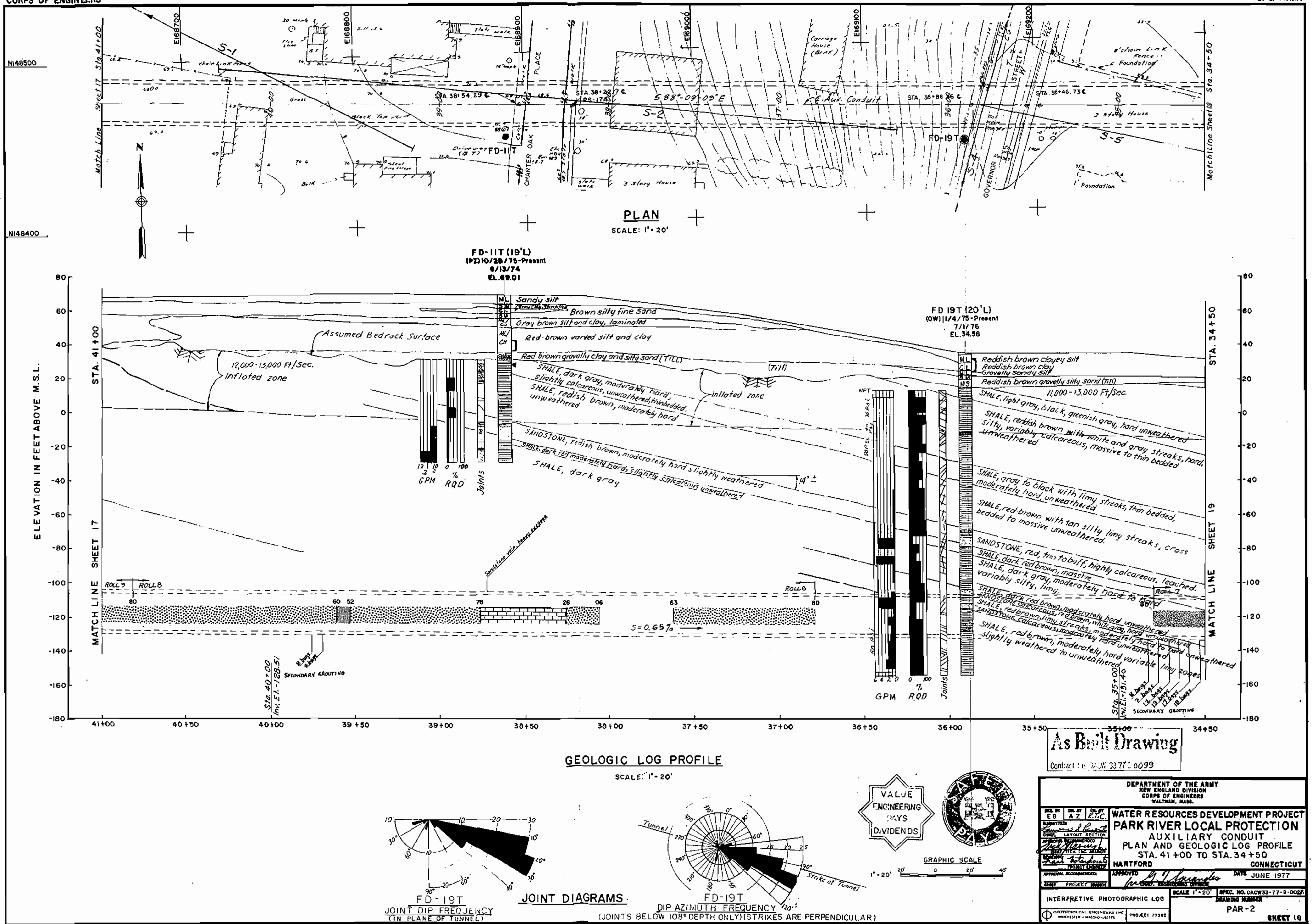


FOR GRAPHIC LEGEND PLEASE REFER TO R.L.I OF THIS REPORT

PLATE 10

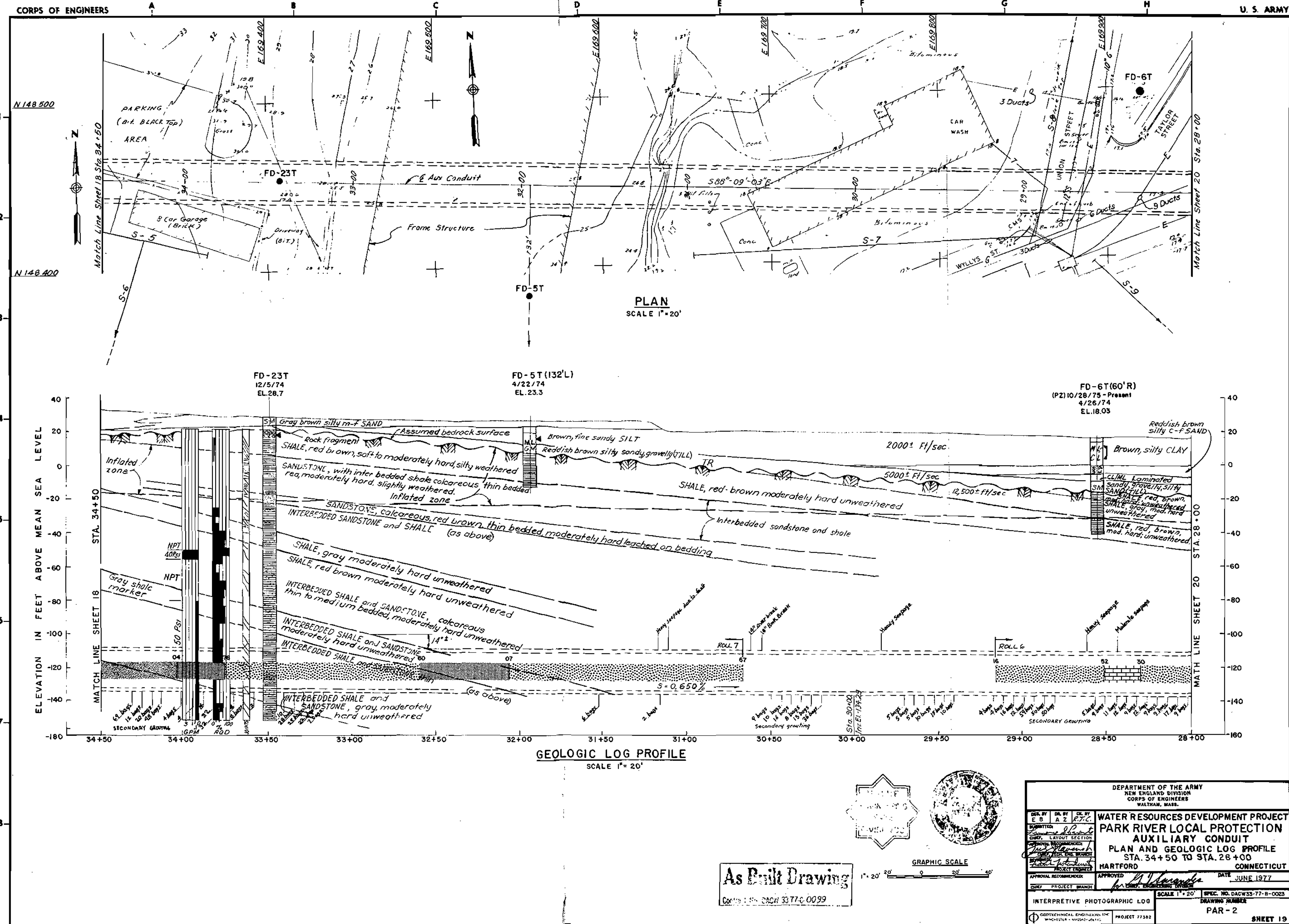
CORPS OF ENGINEERS

U. S. ARMY



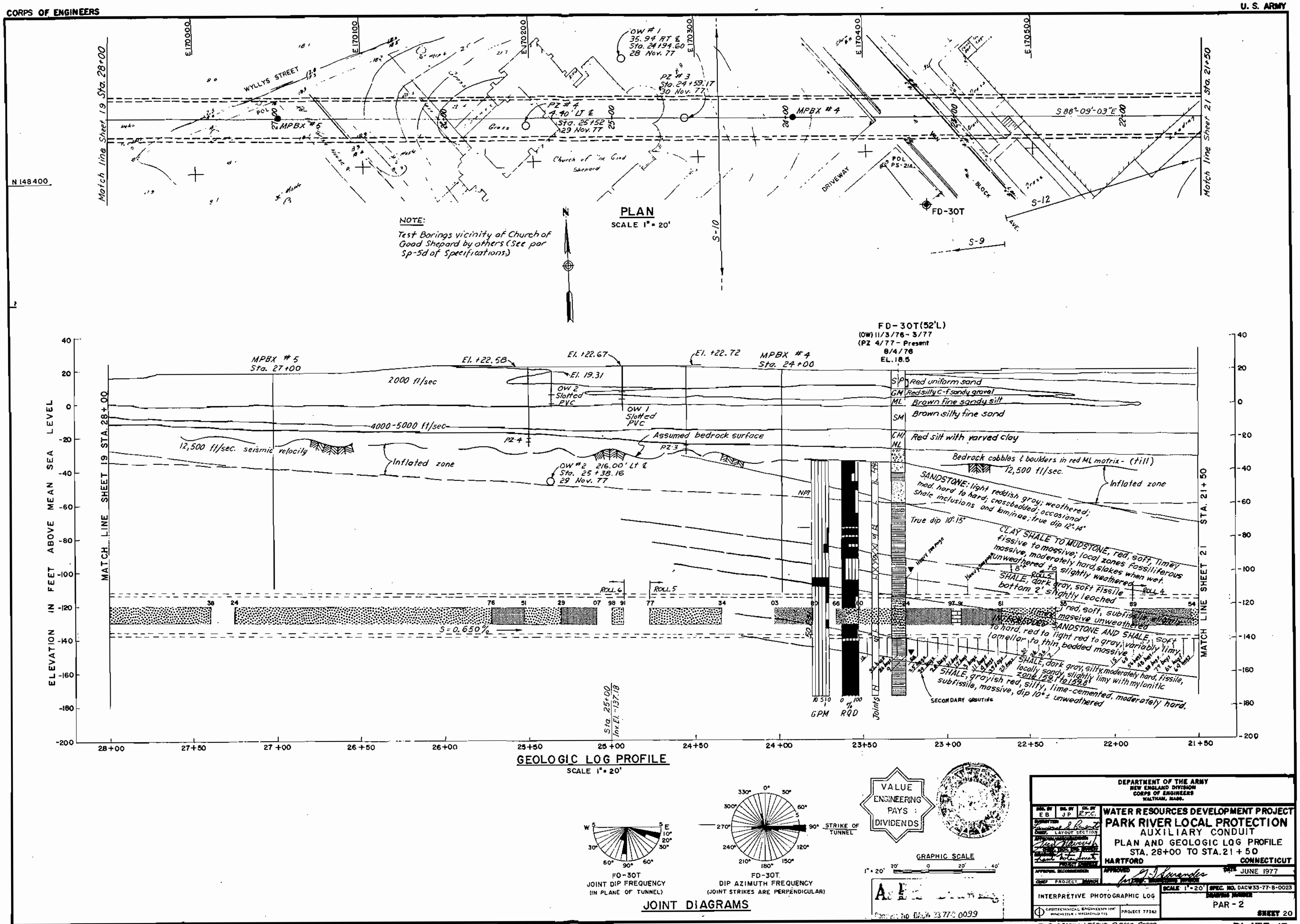
FOR GRAPHIC LEGEND PLEASE REFER TO P. L1 OF THIS REPORT

PLATE 11



FOR GRAPHIC LEGEND PLEASE REFER TO A-1 OF THIS REPORT

PLATE 12



FOR GRAPHIC LEGEND PLEASE REFER TO P. L1 OF THIS REPORT

PLATE 13

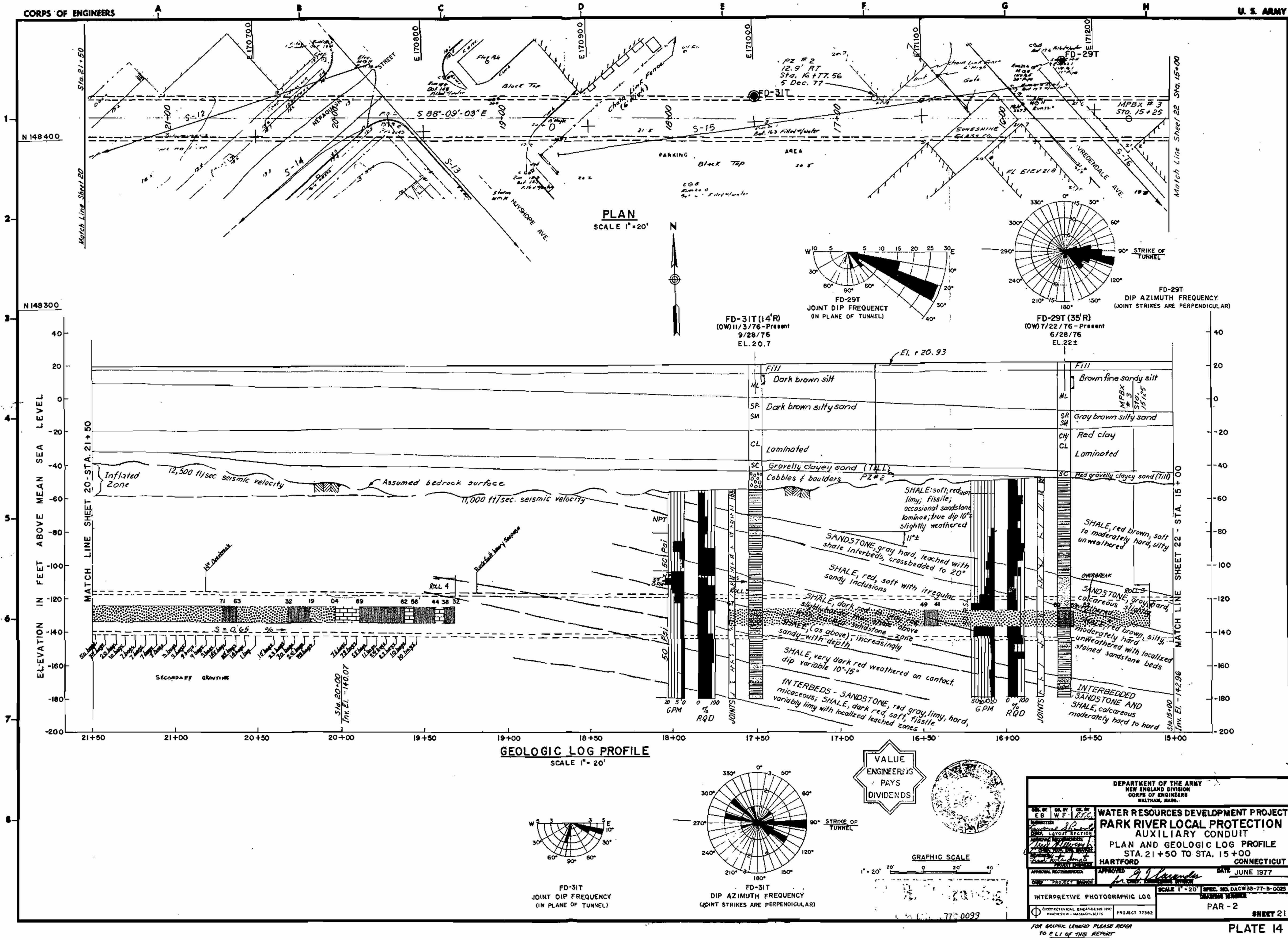
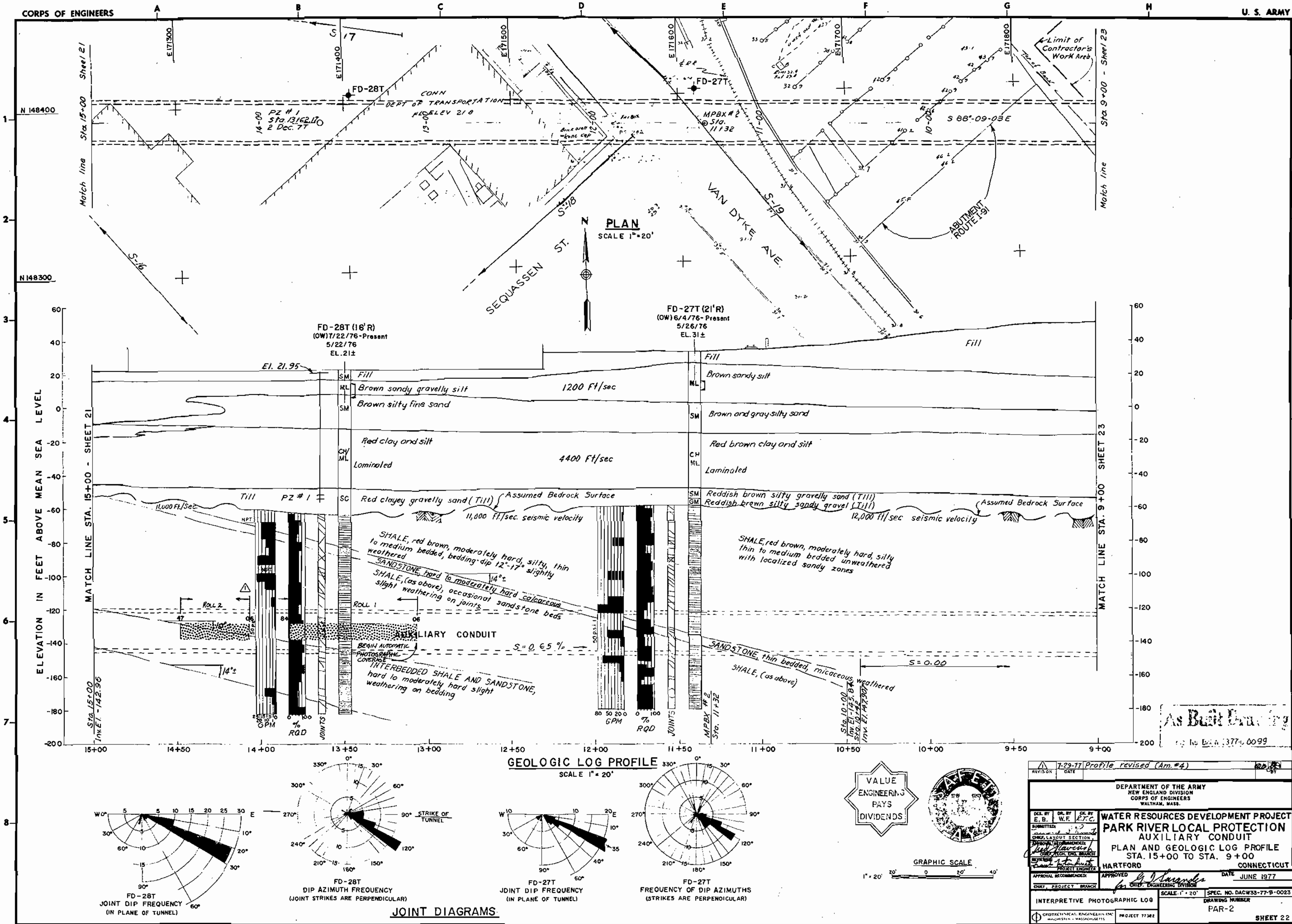
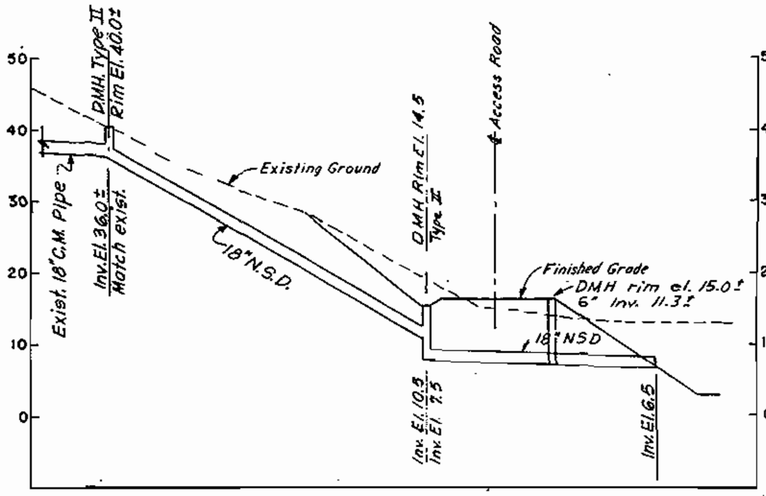

CORPS OF ENGINEERS
U. S. ARMY
PLAN
SCALE 1" = 20'
S 88°-09'-03" E
S-12
S-13
S-14
S-15
S-16
FD-31T
FD-29T
PZ # 2
12.9' RT
Sta. 16+77.56
5 Dec. 77
MPBX # 3
Sta. 15+25
HUYSHOPE AVE.
VREDENDALE AVE.
PARKING
AREA
Black Top
Match Line Sheet 20
Match Line Sheet 22
Sta. 21+50
Sta. 15+00
N 148400
N 148300
E 170700
E 170800
E 170900
E 171000
E 171100
E 171200
FD-29T
JOINT DIP FREQUENCY
(IN PLANE OF TUNNEL)
FD-29T
DIP AZIMUTH FREQUENCY
(JOINT STRIKES ARE PERPENDICULAR)
STRIKE OF TUNNEL
FD-31T (14'R)
(OW) 11/3/76 - Present
9/28/76
EL. 20.7
FD-29T (35'R)
(OW) 7/22/76 - Present
6/28/76
EL. 22±
El. ± 20.93
ELEVATION IN FEET ABOVE MEAN SEA LEVEL
MATCH LINE SHEET 20 - STA. 21+50
MATCH LINE SHEET 22 - STA. 15+00
Fill
Dark brown silt
Dark brown silty sand
Laminated
Gravelly clayey sand (TILL)
Cobbles & boulders
Brown fine sandy silt
Gray brown silty sand
Red clay
Laminated
Red gravelly clayey sand (Till)
Inflated Zone
12,500 ft/sec. seismic velocity
Assumed bedrock surface
11,000 ft/sec. seismic velocity
SHALE: soft; red; limy; fissile; occasional sandstone laminae; true dip 10°±; slightly weathered
SANDSTONE, gray hard, leached with shale interbeds, crossbedded to 20°
SHALE, red, soft with irregular sandy inclusions
SHALE, dark red to gray, slightly hard; like shale above with red sandstone zone
SHALE, (as above), increasingly sandy with depth
SHALE, very dark red weathered on contact, dip variable 10°-15°
INTERBEDS - SANDSTONE, red gray, limy, hard, micaceous; SHALE, dark red, soft, fissile variably limy with localized leached zones
SHALE, red brown, soft to moderately hard, silty unweathered
SANDSTONE, gray, hard, calcareous, slightly weathered
SHALE, red brown, silty, moderately hard, unweathered with localized stained sandstone beds
INTERBEDDED SANDSTONE AND SHALE, calcareous moderately hard to hard
S = 0.65 %
SECONDARY GROUTING
Sta. 20+00
Inv. El. -140.07
Sta. 15+00
Inv. El. -142.96
GPM
% RQD
JOINTS
GEOLOGIC LOG PROFILE
SCALE 1" = 20'
VALUE ENGINEERING PAYS DIVIDENDS
FD-31T
JOINT DIP FREQUENCY
(IN PLANE OF TUNNEL)
FD-31T
DIP AZIMUTH FREQUENCY
(JOINT STRIKES ARE PERPENDICULAR)
GRAPHIC SCALE
1" = 20'
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT
PLAN AND GEOLOGIC LOG PROFILE
STA. 21+50 TO STA. 15+00
HARTFORD CONNECTICUT
DATE JUNE 1977
SCALE 1" = 20'
SPEC. NO. DACW33-77-B-0023
INTERPRETIVE PHOTOGRAPHIC LOG
GEOTECHNICAL ENGINEERS INC.
PROJECT 77392
PAR - 2
SHEET 21
FOR GRAPHIC LEGEND PLEASE REFER TO P. L1 OF THE REPORT

PLATE 14



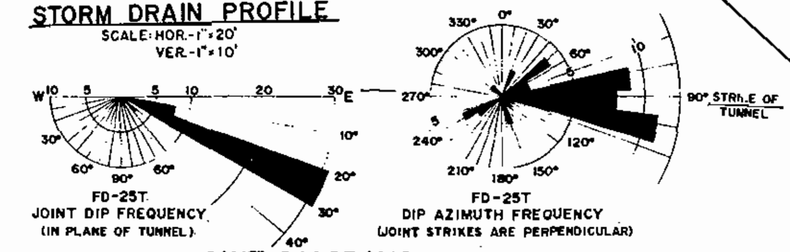
FOR GRAPHIC LEGEND PLEASE REFER TO P L I OF THIS REPORT

PLATE 15

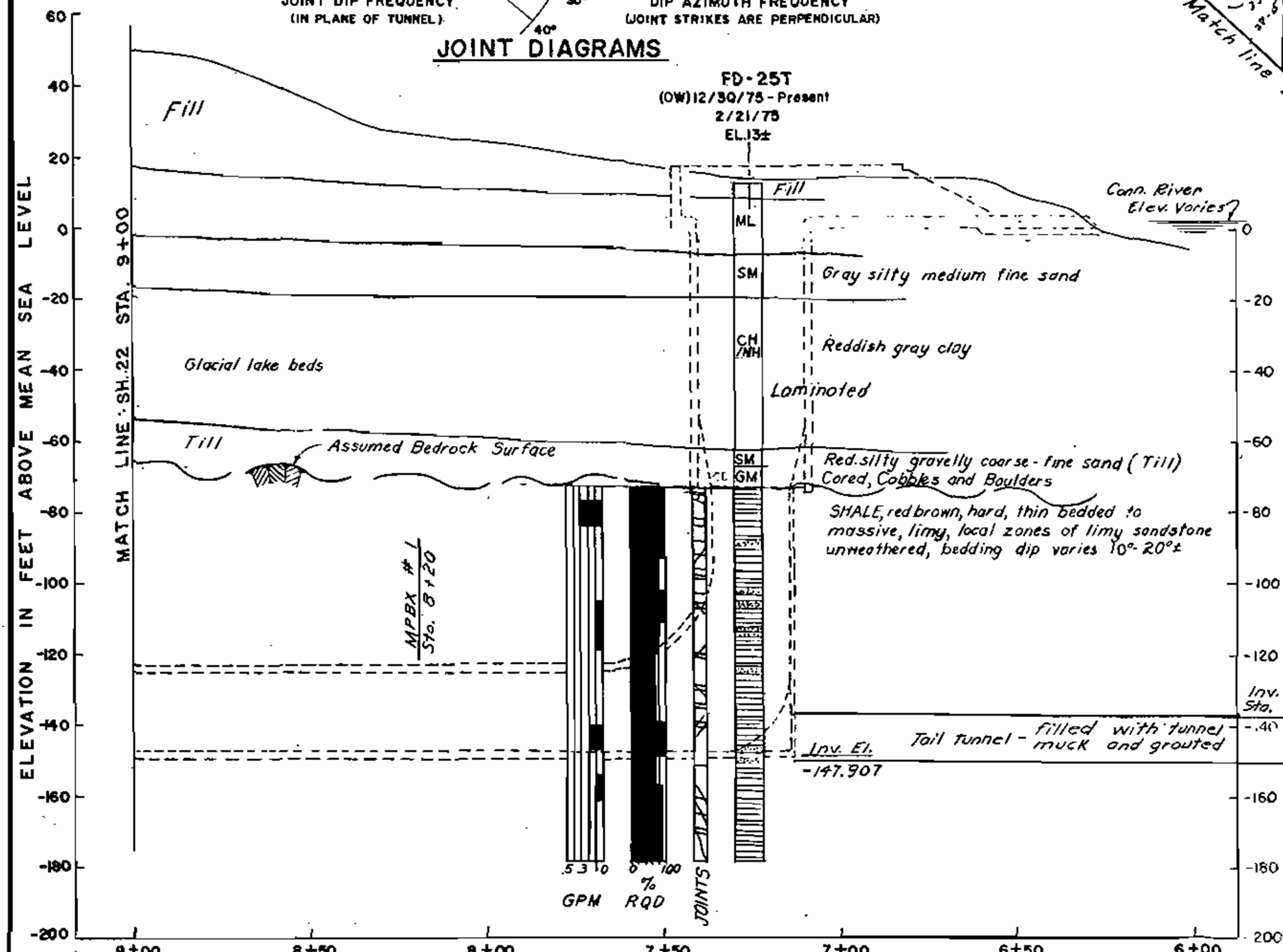


STORM DRAIN PROFILE

SCALE: HOR.-1"=20' VER.-1"=10'

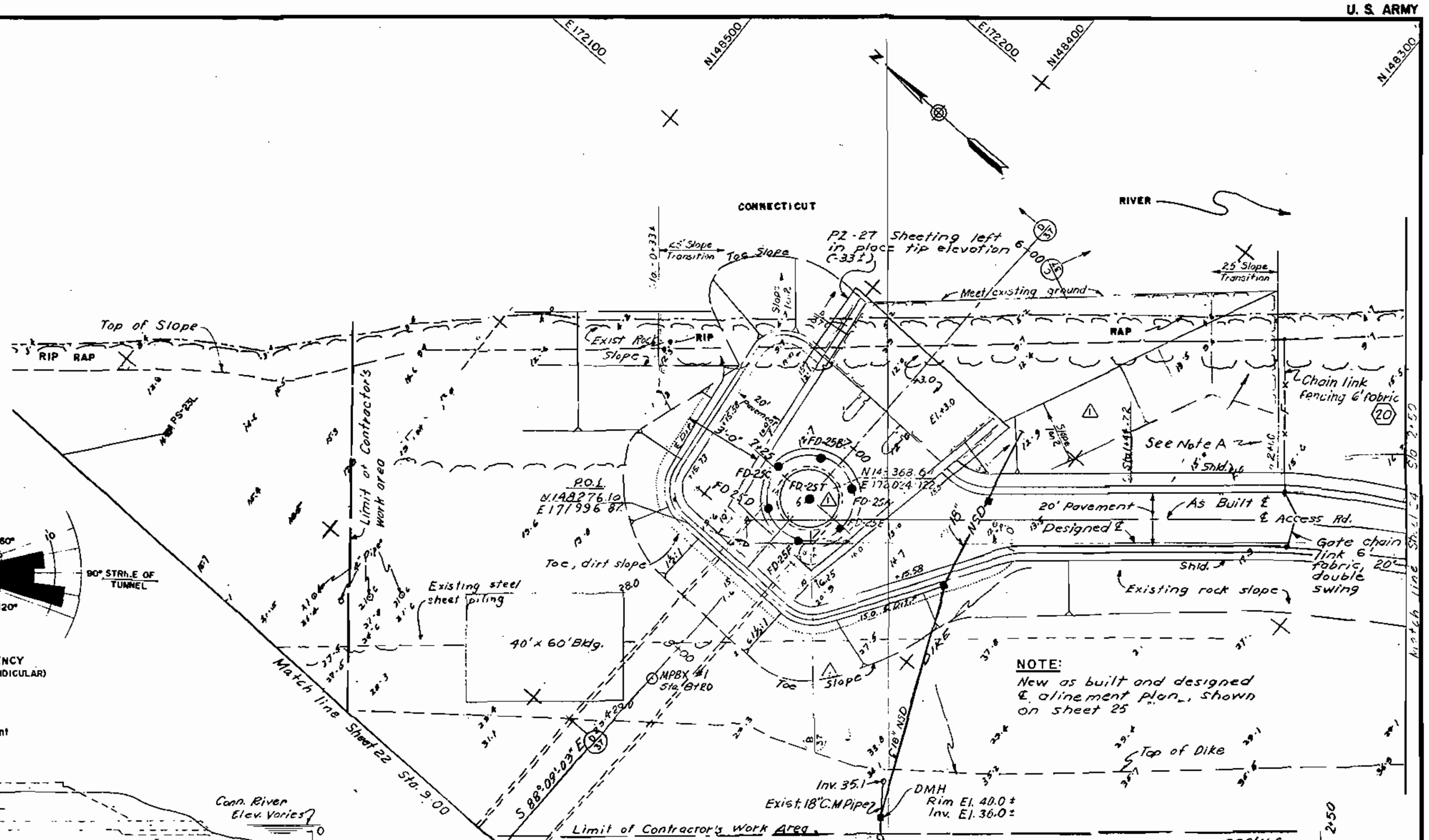


JOINT DIAGRAMS



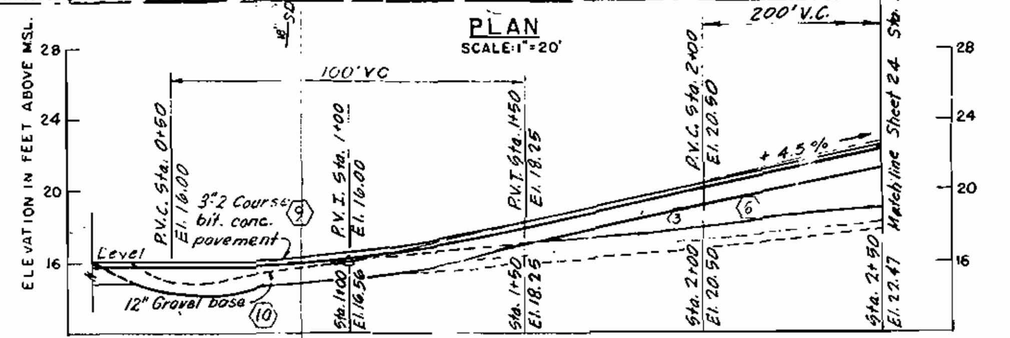
GEOLOGIC LOG PROFILE

SCALE 1" = 20'



PLAN

SCALE: 1" = 20'



PROFILE

SCALE: HORIZ. 1" = 20' VERT. 1" = 4'

NOTE A

The contractor may fill up to 2 acres of land in this area and will not be required to remove the fill providing the final elevation is not above the elevation of the access road and that the area must be covered with 4" of topsoil seeded and sloped to drain. The river slope of the area shall not extend beyond the existing river bank and must be faced with 1' of gravel bedding and 2' of rock slope protection with a maximum slope of 1 on 2. Excavation shall include the removal of rip-rap, concrete caps and wood pile structures, starting at about +3 and their removal to the extent necessary to complete the required excavation.

VALUE ENGINEERING PAYS DIVIDENDS

SAFETY PAYS

No. DACW 33-77-B-0099

GRAPHIC SCALES

REVISION	DATE	DESCRIPTION
		Final field correction
1	7-29-77	Borings added & survey data deleted (Am. #4)

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

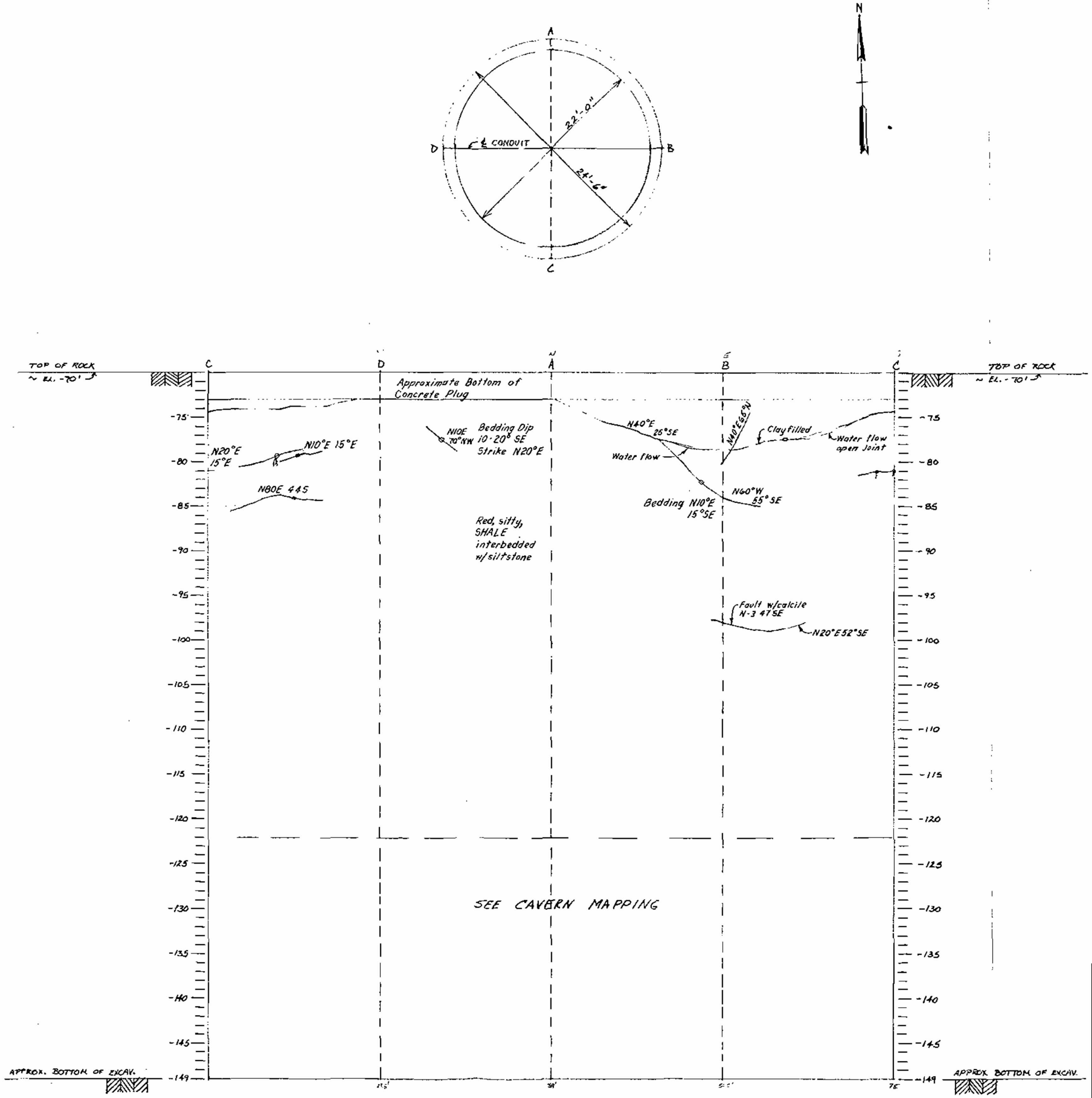
WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT
PLAN AND GEOLOGIC LOG PROFILE
STA. 9+00 TO STA. 6+00
HARTFORD CONNECTICUT

DATE JUNE 1977

SCALE AS SHOWN | SPEC. NO. DACW 33-77-B-0025

DRAWING NUMBER PAR-2

SHEET 23



LEGEND

Strike & Dip of Joints

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

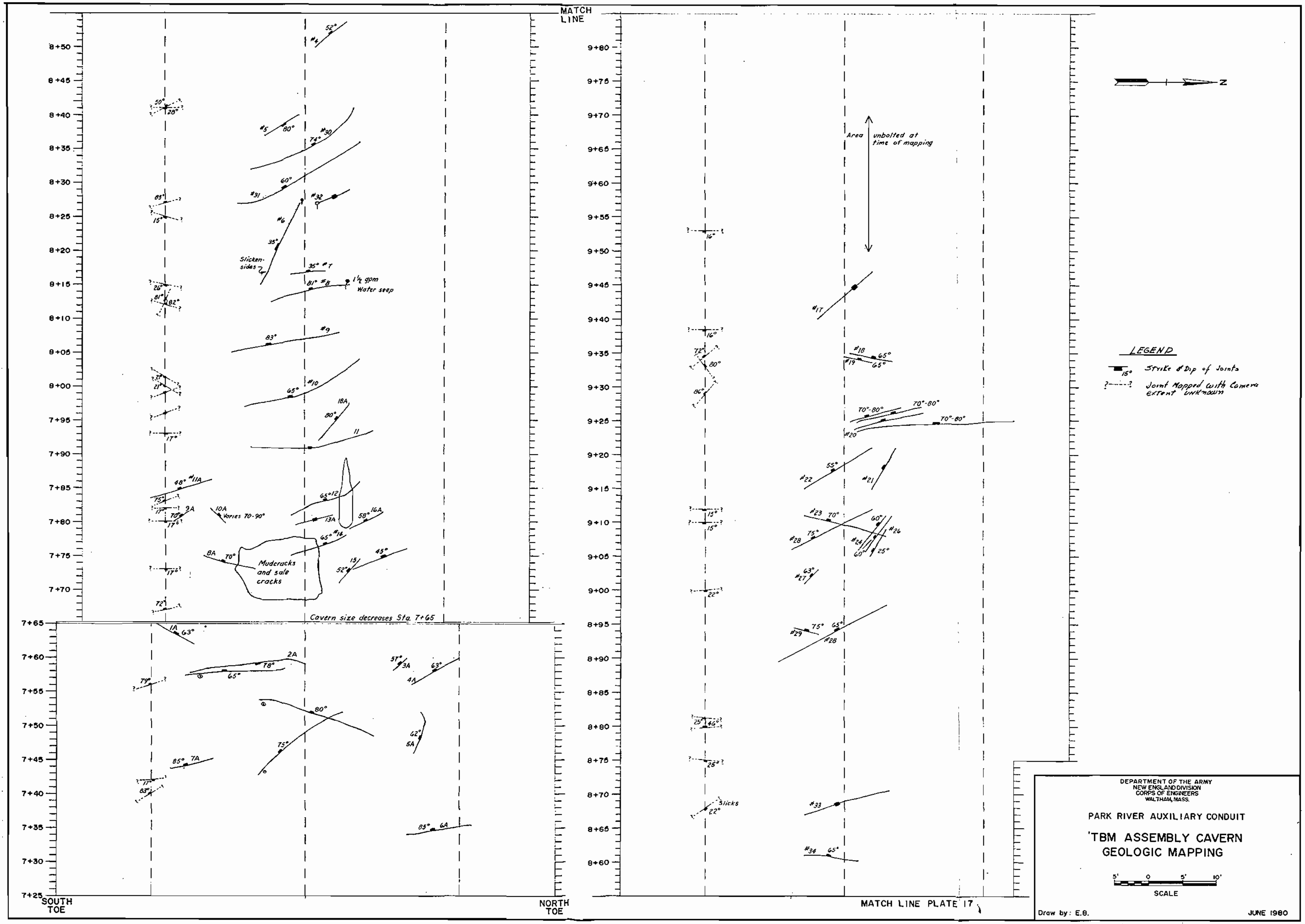
PARK RIVER AUXILIARY CONDUIT

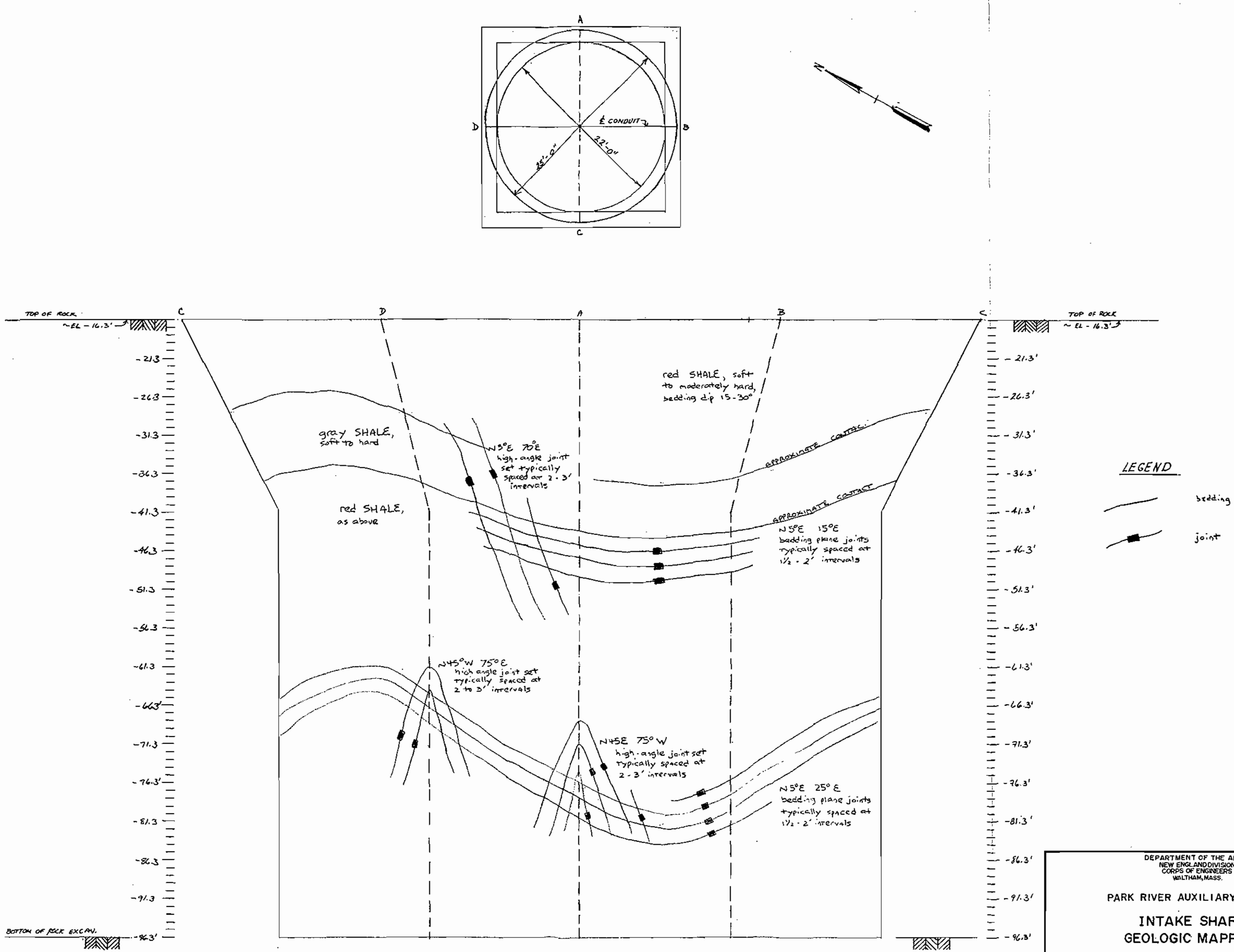
OUTLET SHAFT
GEOLOGIC MAPPING

5' 0 5' 10'
SCALE

Drawn by: E.B. JUNE 1982

PLATE 17


MATCH LINE
SOUTH TOE
NORTH TOE
MATCH LINE PLATE 17
Cavern size decreases Sta. 7+65
Mudcracks and sale cracks
Slicken-sides
1½ gpm Water seep
Area unbolted at time of mapping
Slicks
10A Varies 70-90°
LEGEND
Strike & Dip of Joints
Joint Mapped with Camera extent unknown
SCALE
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
PARK RIVER AUXILIARY CONDUIT
TBM ASSEMBLY CAVERN
GEOLOGIC MAPPING
Draw by: E.S.
JUNE 1980



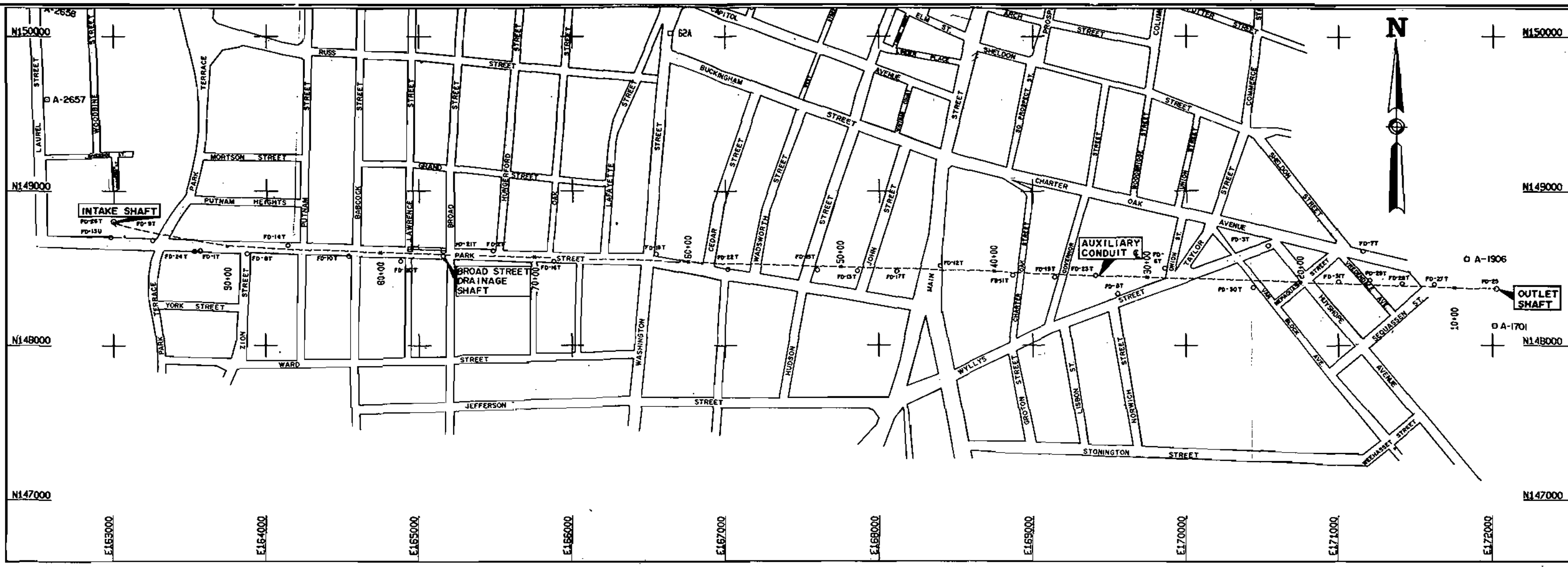
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

PARK RIVER AUXILIARY CONDUIT

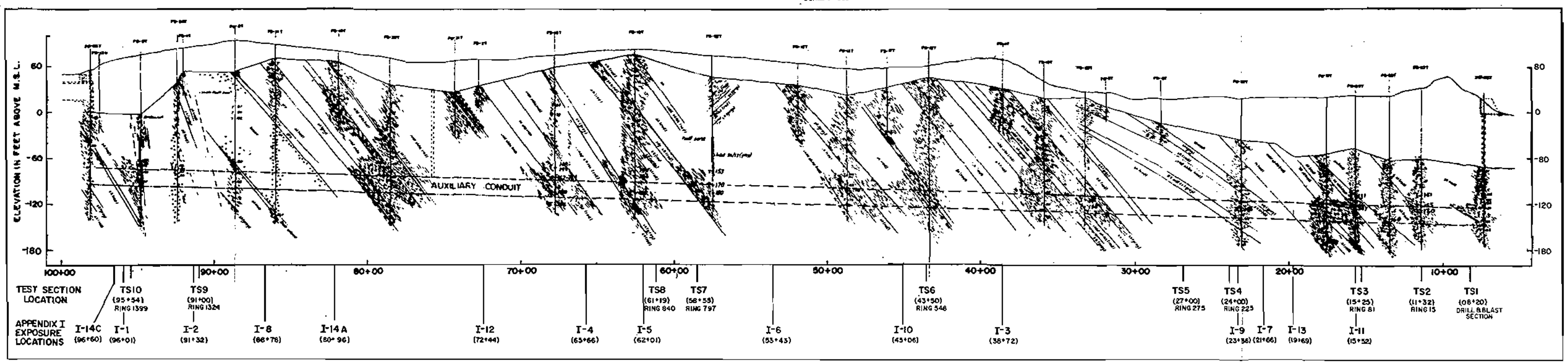
INTAKE SHAFT
GEOLOGIC MAPPING

5' 0 5' 10'
SCALE

Draw by: E.B. JUNE 1980



PLAN
SCALE 1" = 300'



PROFILE
VERTICAL 1" = 60 FT.
HORIZONTAL 1" = 300 FT.

Note
Base Plans obtained from COE. Drawings PAR 2, sheets 3 and 90.

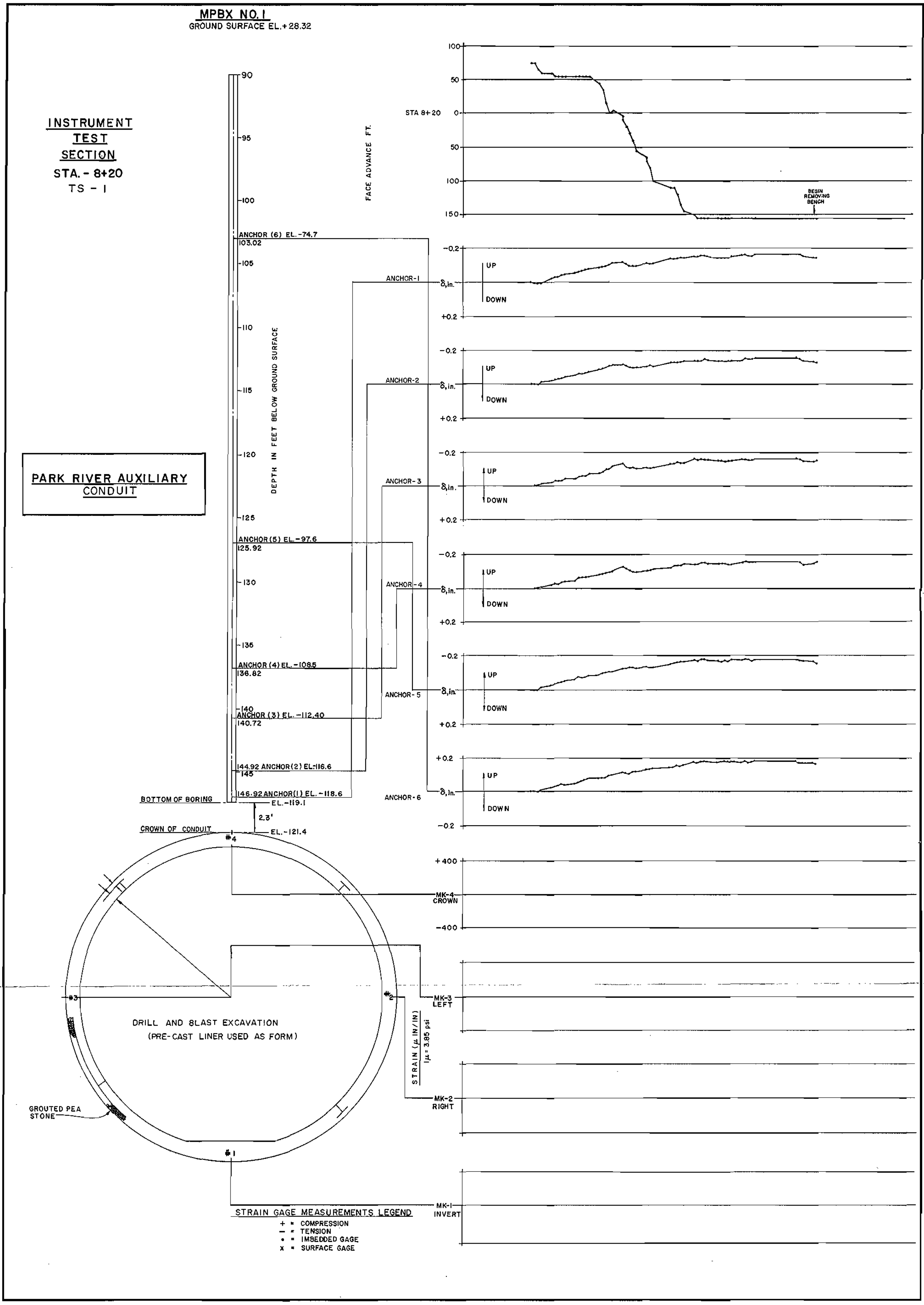
ROGER J. AU & SON, INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL HARTFORD, CT.	PROJECT PLAN AND PROFILE
GEOTECHNICAL ENGINEERS INC. WINCHESTER, MASSACHUSETTS	PROJECT 77382	OCTOBER, 1980 FIG. 2

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

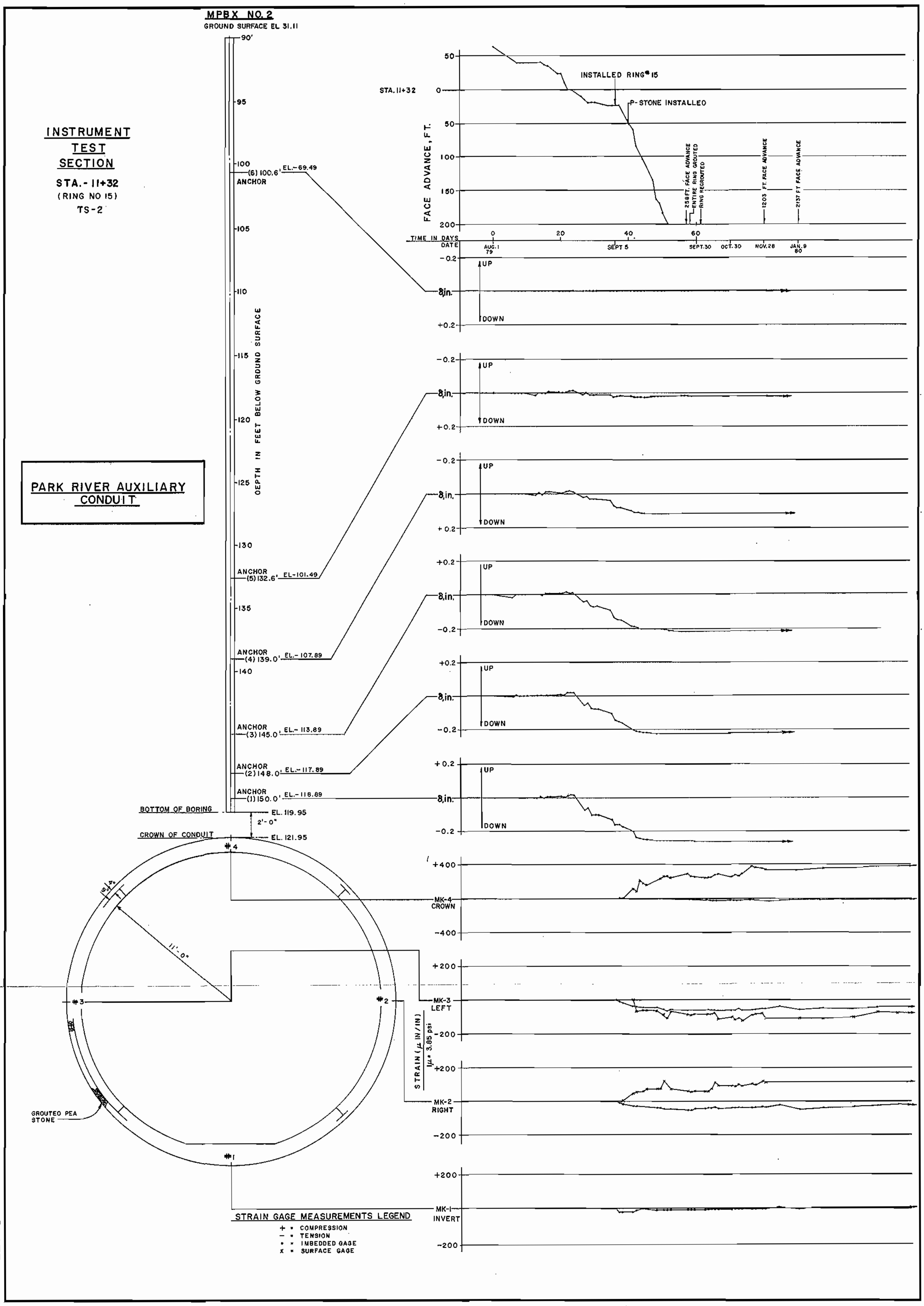
PARK RIVER AUXILIARY CONDUIT

INSTRUMENTATION LOCATION

PLAN AND PROFILE



MPBX NO. 1
GROUND SURFACE EL. + 28.32
INSTRUMENT TEST SECTION
STA. - 8+20
TS - 1
PARK RIVER AUXILIARY CONDUIT
FACE ADVANCE FT.
STA 8+20
BEGIN REMOVING BENCH
DEPTH IN FEET BELOW GROUND SURFACE
ANCHOR (6) EL. -74.7
103.02
ANCHOR (5) EL. -97.6
125.92
ANCHOR (4) EL. -108.5
136.82
ANCHOR (3) EL. -112.40
140.72
144.92 ANCHOR (2) EL. -116.6
146.92 ANCHOR (1) EL. -118.6
BOTTOM OF BORING
EL. -119.1
2.3'
CROWN OF CONDUIT
EL. -121.4
ANCHOR-1
ANCHOR-2
ANCHOR-3
ANCHOR-4
ANCHOR-5
ANCHOR-6
δ, in.
UP
DOWN
MK-4 CROWN
MK-3 LEFT
MK-2 RIGHT
MK-1 INVERT
STRAIN (μ IN/IN) 1μ = 3.85 psi
DRILL AND BLAST EXCAVATION
(PRE-CAST LINER USED AS FORM)
GROUTED PEA STONE
STRAIN GAGE MEASUREMENTS LEGEND
+ = COMPRESSION
− = TENSION
• = IMBEDDED GAGE
x = SURFACE GAGE



MPBX NO. 2
GROUND SURFACE EL 31.11
INSTRUMENT TEST SECTION
STA. - 11+32
(RING NO 15)
TS-2
PARK RIVER AUXILIARY CONDUIT
DEPTH IN FEET BELOW GROUND SURFACE
90'
95
100
105
110
115
120
125
130
135
140
(6) 100.6' ANCHOR EL.-69.49
ANCHOR (5) 132.6' EL.-101.49
ANCHOR (4) 139.0' EL.-107.89
ANCHOR (3) 145.0' EL.-113.89
ANCHOR (2) 148.0' EL.-117.89
ANCHOR (1) 150.0' EL.-118.89
BOTTOM OF BORING
EL. 119.95
2'-0"
CROWN OF CONDUIT
EL. 121.95
STA. 11+32
FACE ADVANCE, FT.
INSTALLED RING# 15
P-STONE INSTALLED
258 FT. FACE ADVANCE
ENTIRE RING GROUTED
RING REGROUTED
1203 FT. FACE ADVANCE
2137 FT. FACE ADVANCE
TIME IN DAYS
DATE
AUG. 1 79
SEPT. 5
SEPT. 30
OCT. 30
NOV. 28
JAN. 9 80
UP
DOWN
℄ in.
11'-0"
GROUTED PEA STONE
#1
#2
#3
#4
MK-4 CROWN
MK-3 LEFT
MK-2 RIGHT
MK-1 INVERT
STRAIN (μ IN/IN)
1μ = 3.85 psi
STRAIN GAGE MEASUREMENTS LEGEND
+ = COMPRESSION
− = TENSION
• = IMBEDDED GAGE
x = SURFACE GAGE

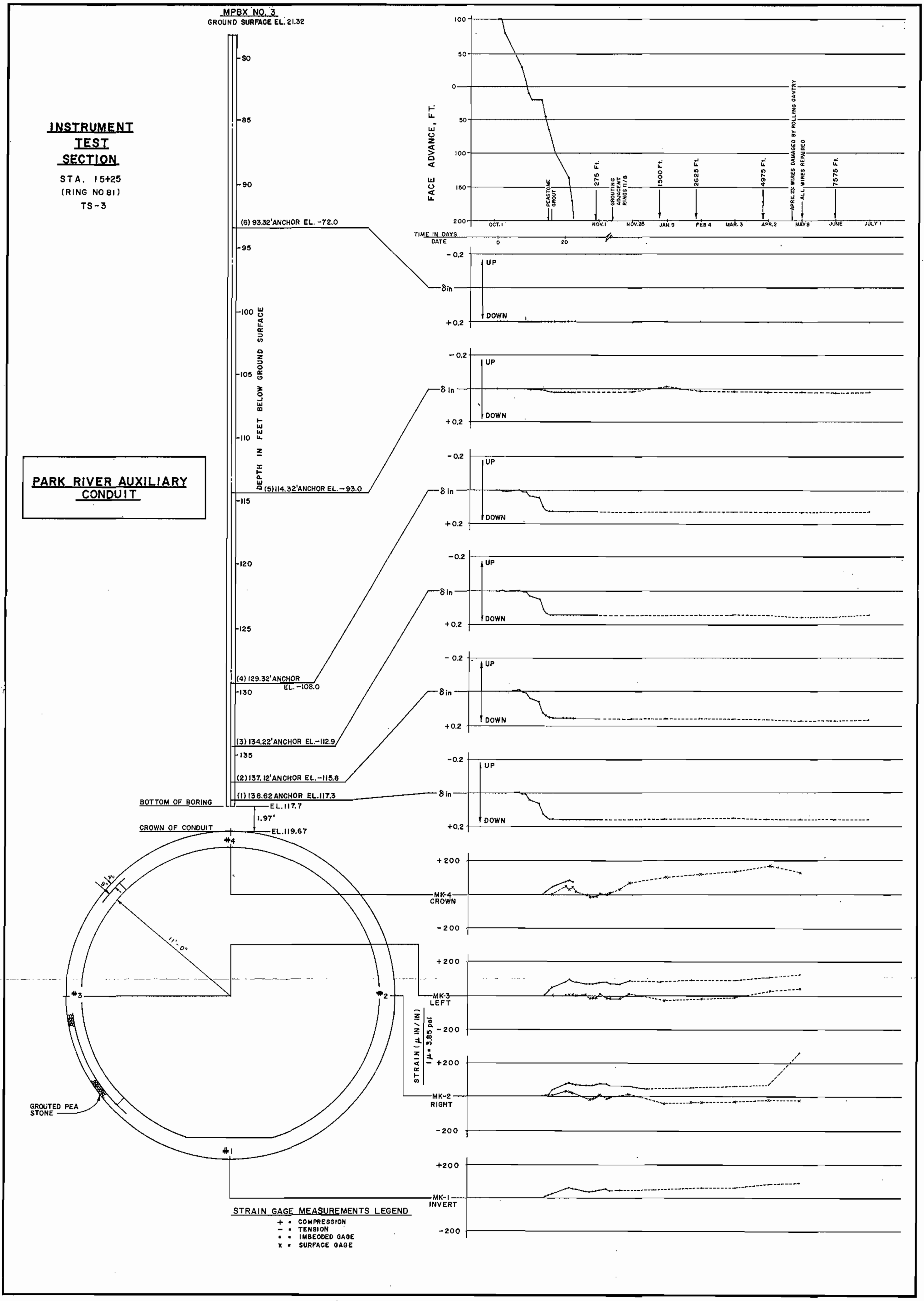

INSTRUMENT TEST SECTION
STA. 15+25
(RING NO 81)
TS-3
PARK RIVER AUXILIARY CONDUIT
MPBX NO. 3
GROUND SURFACE EL. 21.32
DEPTH IN FEET BELOW GROUND SURFACE
(6) 93.32' ANCHOR EL. -72.0
(5) 114.32' ANCHOR EL. -93.0
(4) 129.32' ANCHOR EL. -108.0
(3) 134.22' ANCHOR EL. -112.9
(2) 137.12' ANCHOR EL. -115.8
(1) 138.62 ANCHOR EL. 117.3
BOTTOM OF BORING
EL. 117.7
1.97'
CROWN OF CONDUIT
EL. 119.67
11'-0"
GROUTED PEA STONE
FACE ADVANCE, FT.
PEASTONE GROUT
275 Ft.
GROUTING ADJACENT RINGS 11/8
1500 Ft.
2625 Ft.
4975 Ft.
APRIL 23-WIRES DAMAGED BY ROLLING GANTRY
ALL WIRES REPAIRED
7575 Ft.
OCT. 1
NOV. 1
NOV. 28
JAN. 9
FEB 4
MAR. 3
APR. 2
MAY 8
JUNE
JULY 1
TIME IN DAYS
DATE
UP
DOWN
δ in
MK-4 CROWN
MK-3 LEFT
MK-2 RIGHT
MK-1 INVERT
STRAIN (μ IN/IN)
1 μ = 3.85 psi
STRAIN GAGE MEASUREMENTS LEGEND
+ = COMPRESSION
− = TENSION
• = IMBEDDED GAGE
x = SURFACE GAGE

PLATE 23

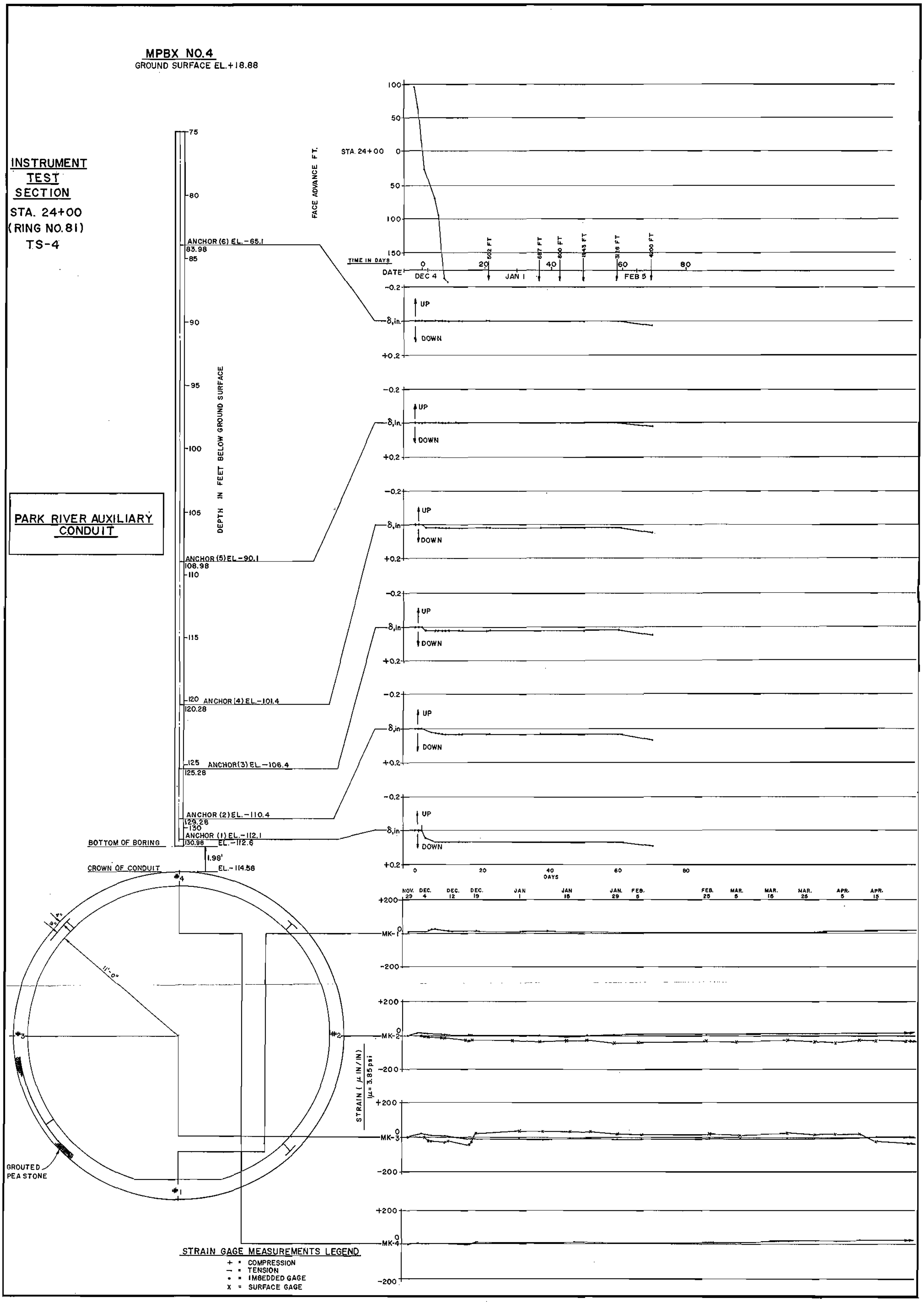
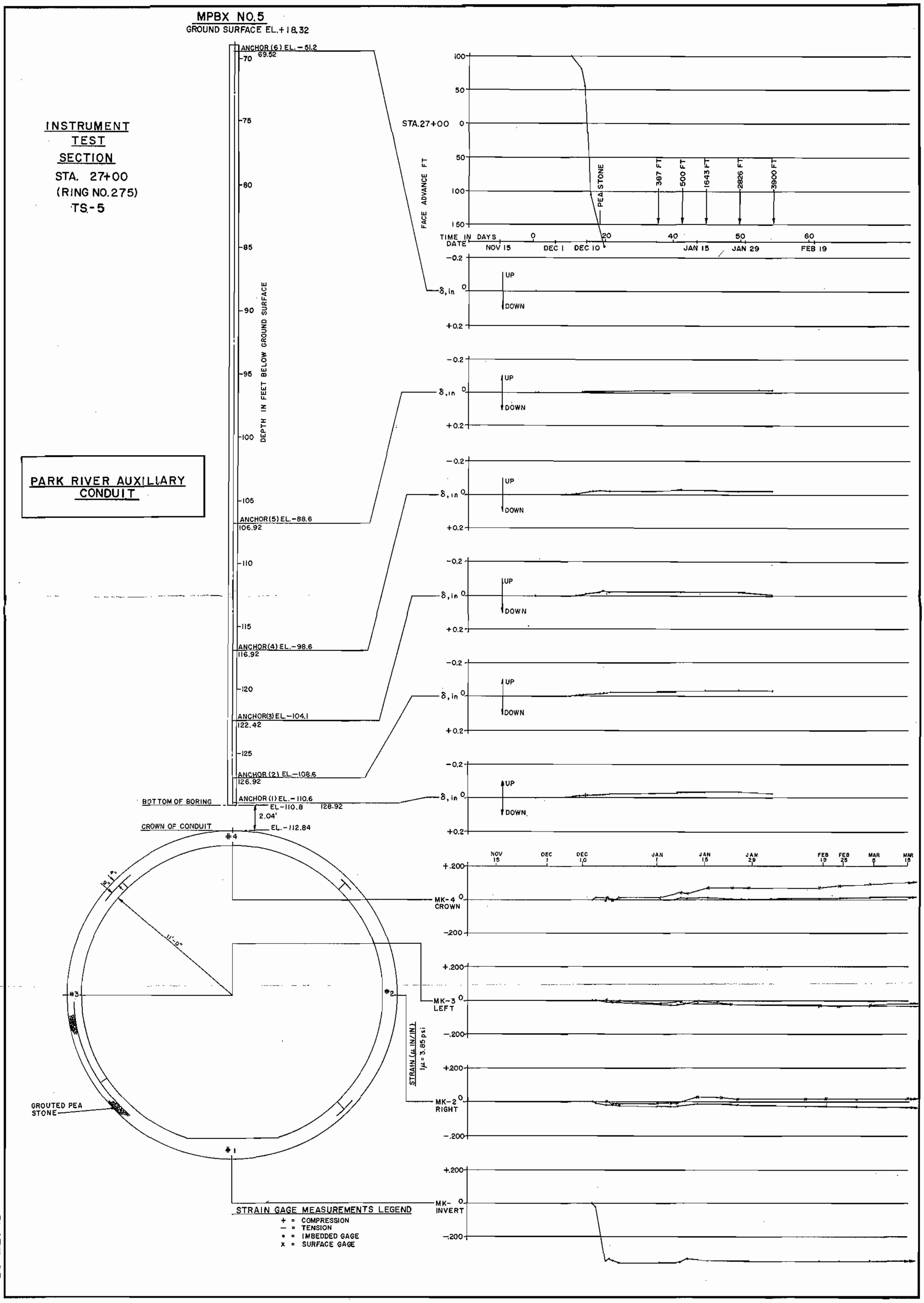
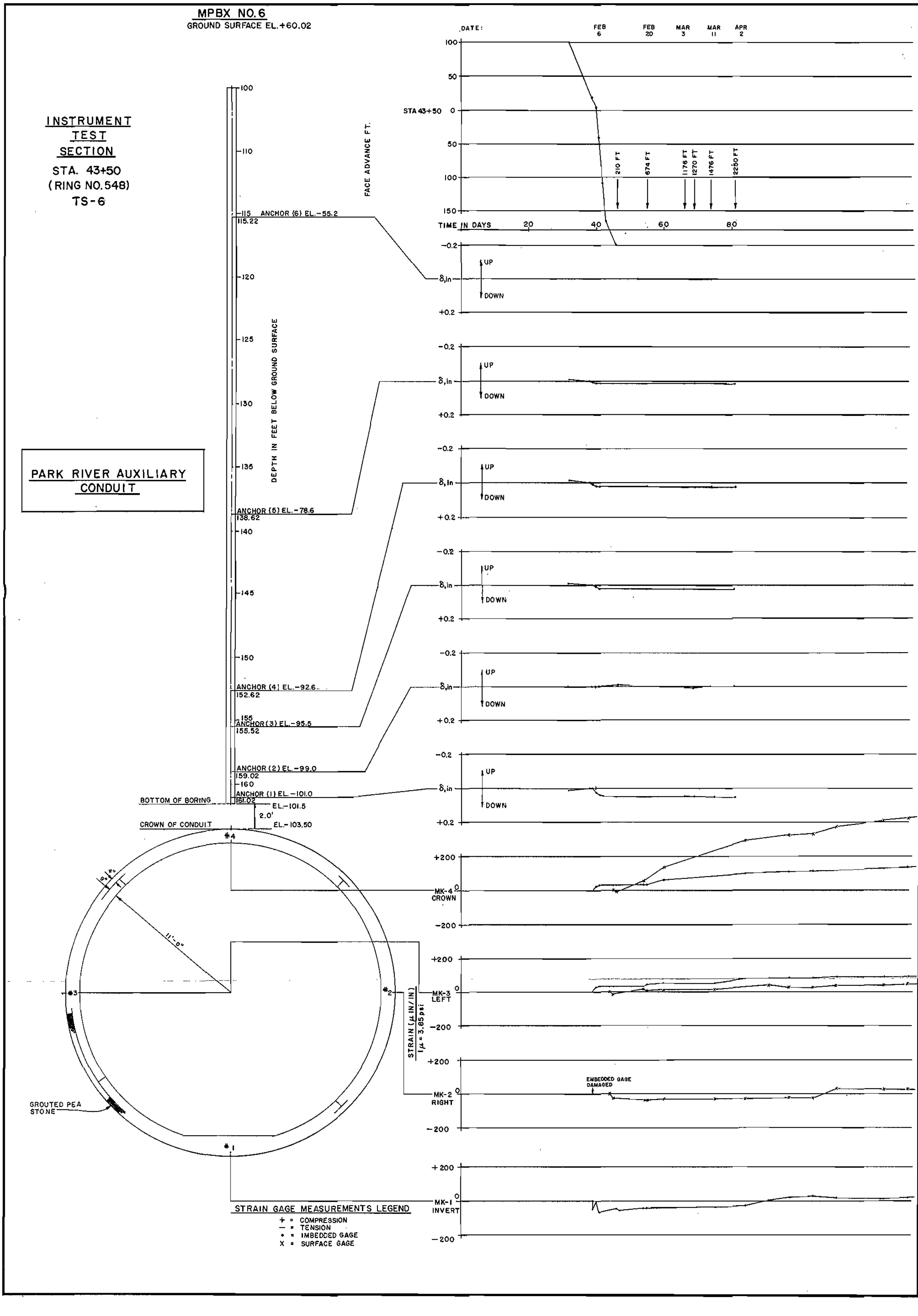

MPBX NO.4
GROUND SURFACE EL.+18.88
INSTRUMENT TEST SECTION
STA. 24+00
(RING NO.81)
TS-4
PARK RIVER AUXILIARY CONDUIT
DEPTH IN FEET BELOW GROUND SURFACE
ANCHOR (6) EL.-65.1
83.98
ANCHOR (5) EL.-90.1
108.98
ANCHOR (4) EL.-101.4
120.28
ANCHOR (3) EL.-106.4
125.28
ANCHOR (2) EL.-110.4
129.28
ANCHOR (1) EL.-112.1
130.98
BOTTOM OF BORING
EL.-112.6
1.98'
CROWN OF CONDUIT
EL.-114.58
FACE ADVANCE FT.
STA. 24+00
TIME IN DAYS
DATE
DEC 4
JAN 1
FEB 5
502 FT
687 FT
800 FT
1143 FT
1316 FT
1400 FT
UP
DOWN
δ, in.
DAYS
NOV. 29
DEC. 4
DEC. 12
DEC. 19
JAN 1
JAN 16
JAN. 29
FEB. 5
FEB. 25
MAR. 5
MAR. 16
MAR. 25
APR. 5
APR. 15
MK-1
MK-2
MK-3
MK-4
STRAIN (μ IN/IN)
1μ = 3.85 psi
11'-0"
GROUTED PEA STONE
STRAIN GAGE MEASUREMENTS LEGEND
+ = COMPRESSION
− = TENSION
• = IMBEDDED GAGE
x = SURFACE GAGE

PLATE 24


MPBX NO.5
GROUND SURFACE EL.+18.32
INSTRUMENT TEST SECTION
STA. 27+00
(RING NO.275)
TS-5
PARK RIVER AUXILIARY CONDUIT
DEPTH IN FEET BELOW GROUND SURFACE
ANCHOR (6) EL.−51.2
69.52
ANCHOR (5) EL.−88.6
106.92
ANCHOR (4) EL.−98.6
116.92
ANCHOR (3) EL.−104.1
122.42
ANCHOR (2) EL.−108.6
126.92
ANCHOR (1) EL.−110.6
BOTTOM OF BORING
EL−110.8
128.92
2.04'
CROWN OF CONDUIT
EL.−112.84
STA.27+00
FACE ADVANCE FT
PEA STONE
387 FT
500 FT
1643 FT
2826 FT
3900 FT
TIME IN DAYS
DATE
NOV 15
DEC 1
DEC 10
JAN 15
JAN 29
FEB 19
δ, in
UP
DOWN
11'-0"
GROUTED PEA STONE
MK-4 CROWN
MK-3 LEFT
MK-2 RIGHT
MK- INVERT
STRAIN (μ IN/IN)
1μ = 3.85 psi
STRAIN GAGE MEASUREMENTS LEGEND
+ = COMPRESSION
− = TENSION
• = IMBEDDED GAGE
x = SURFACE GAGE


MPBX NO. 6
GROUND SURFACE EL. +60.02
INSTRUMENT TEST SECTION
STA. 43+50
(RING NO. 548)
TS-6
PARK RIVER AUXILIARY CONDUIT
DEPTH IN FEET BELOW GROUND SURFACE
ANCHOR (6) EL. -55.2
115.22
ANCHOR (5) EL. -78.6
138.62
ANCHOR (4) EL. -92.6
152.62
ANCHOR (3) EL. -95.5
155.52
ANCHOR (2) EL. -99.0
159.02
ANCHOR (1) EL. -101.0
161.02
BOTTOM OF BORING
EL. -101.5
2.0'
CROWN OF CONDUIT
EL. -103.50
11'-0"
GROUTED PEA STONE
DATE:
FEB 6
FEB 20
MAR 3
MAR 11
APR 2
FACE ADVANCE FT.
STA 43+50
210 FT
674 FT
1176 FT
1270 FT
1476 FT
2250 FT
TIME IN DAYS
UP
DOWN
δ, in
EMBEDDED GAGE DAMAGED
MK-4 CROWN
MK-3 LEFT
MK-2 RIGHT
MK-1 INVERT
STRAIN (μ IN/IN)
1 μ = 3.85 psi
STRAIN GAGE MEASUREMENTS LEGEND
+ = COMPRESSION
− = TENSION
• = IMBEDDED GAGE
x = SURFACE GAGE

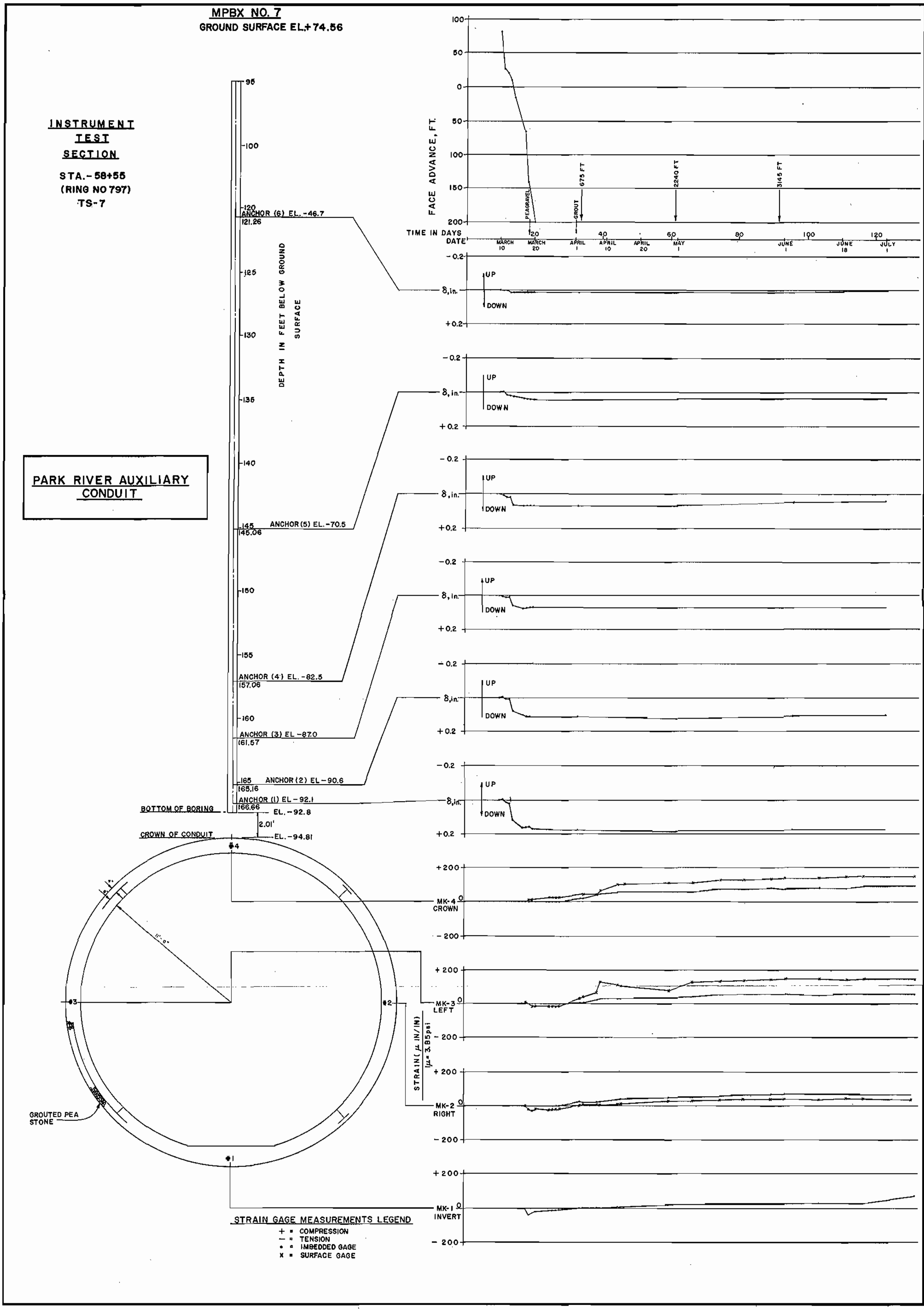
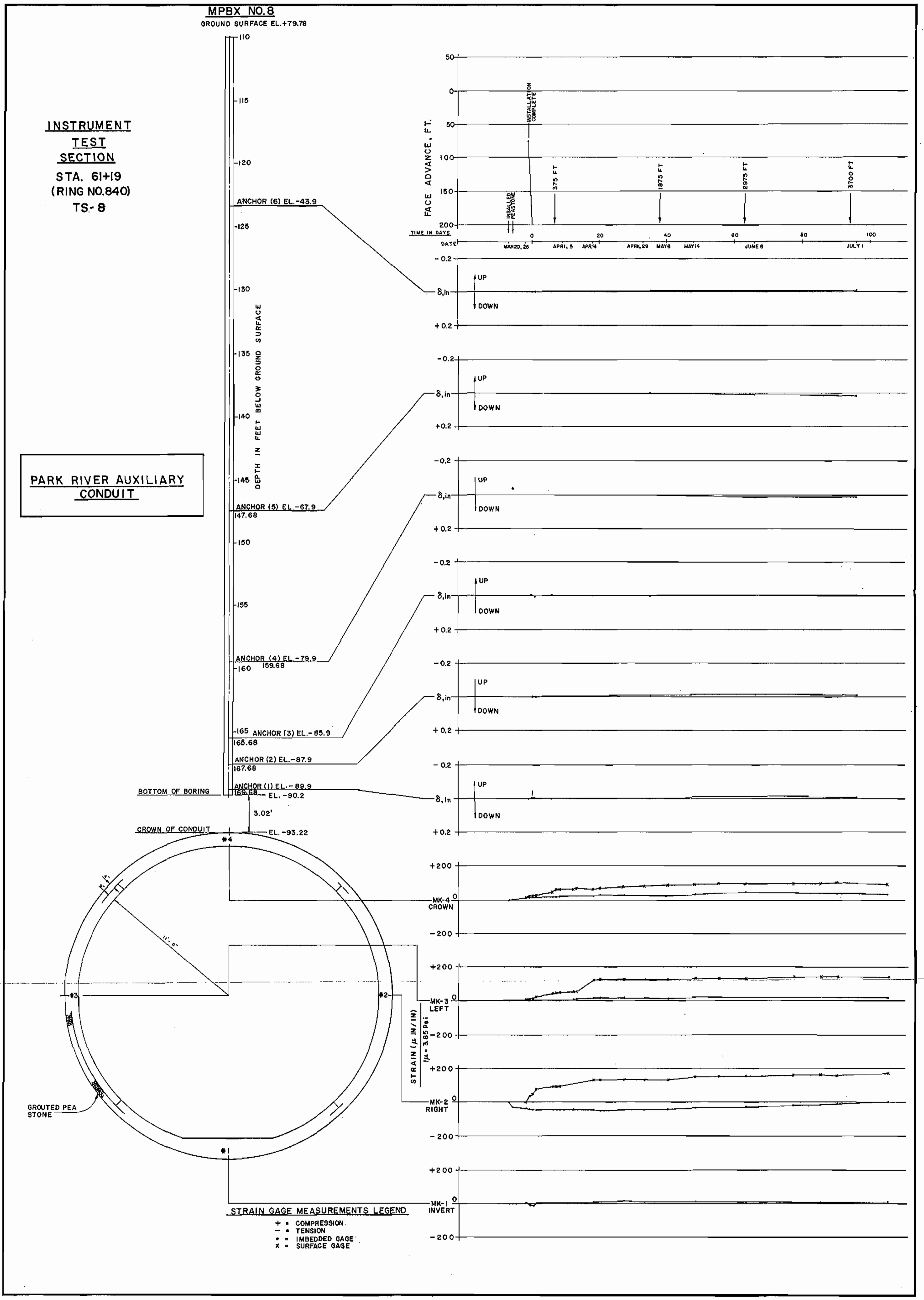
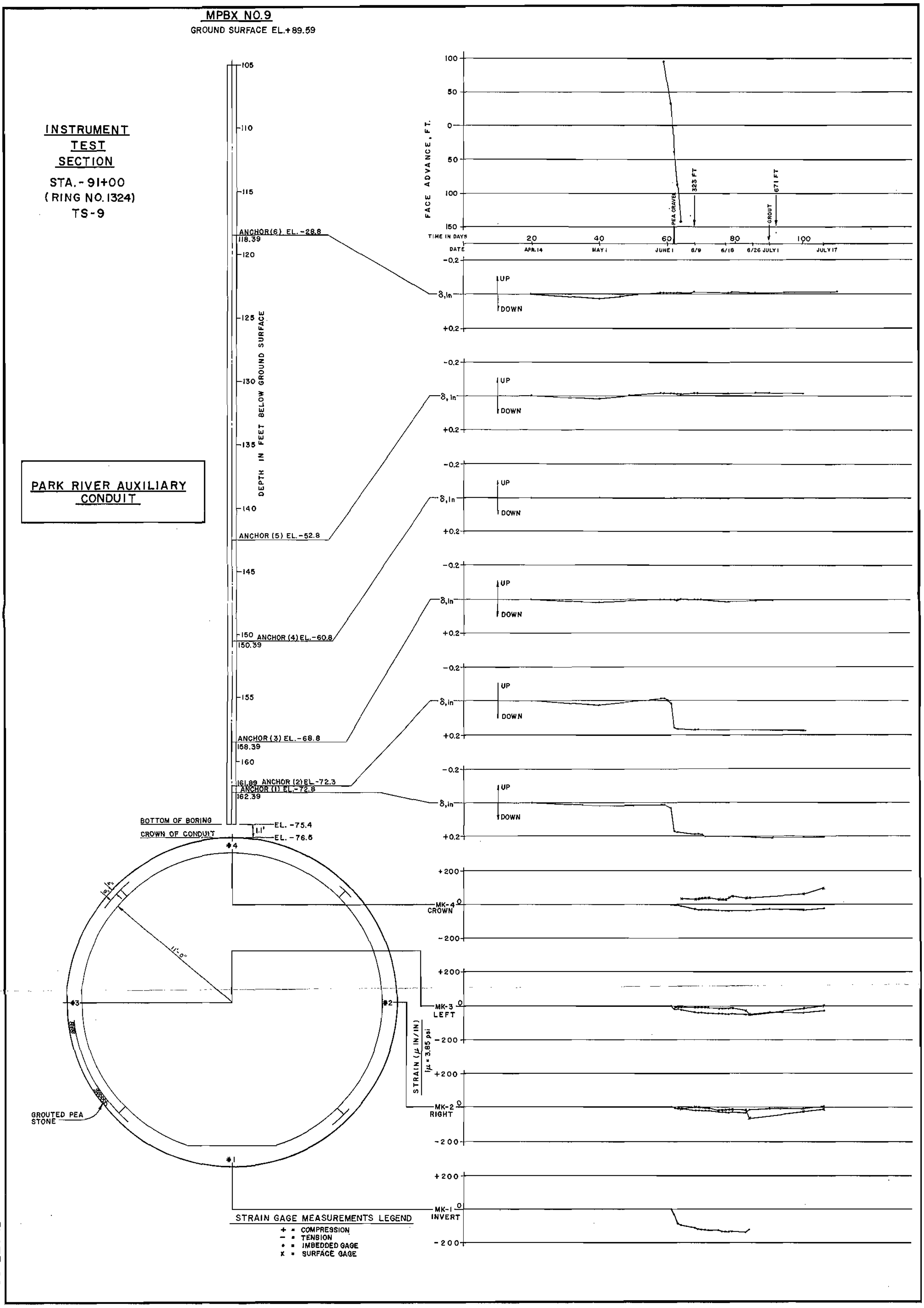
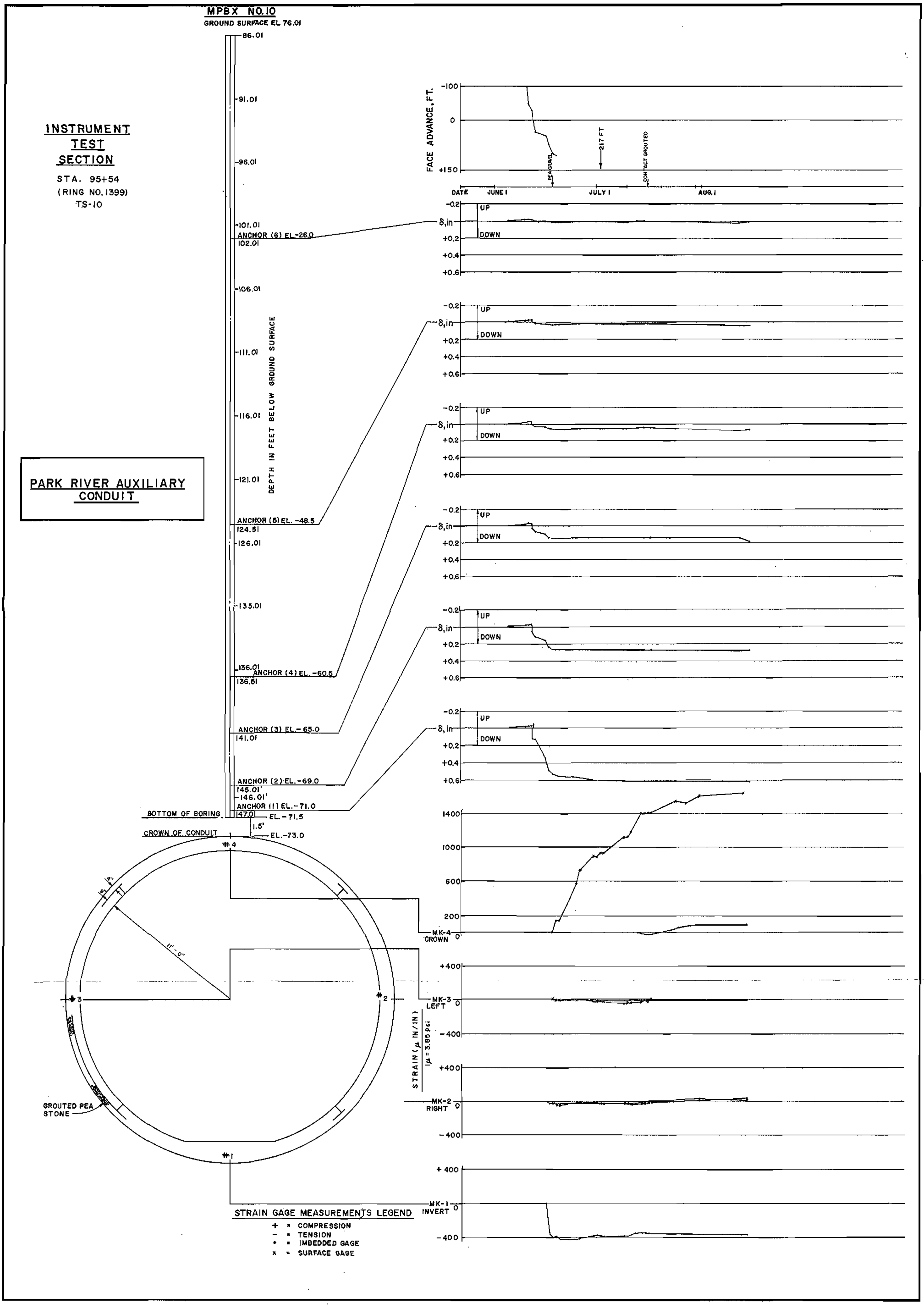

MPBX NO. 7
GROUND SURFACE EL.+74.56
INSTRUMENT TEST SECTION
STA.- 58+55
(RING NO 797)
TS-7
PARK RIVER AUXILIARY CONDUIT
DEPTH IN FEET BELOW GROUND SURFACE
ANCHOR (6) EL.-46.7
121.26
ANCHOR (5) EL.-70.5
145.06
ANCHOR (4) EL.-82.5
157.06
ANCHOR (3) EL-87.0
161.57
ANCHOR (2) EL-90.6
165.16
ANCHOR (1) EL-92.1
166.66
BOTTOM OF BORING
EL.-92.8
2.01'
CROWN OF CONDUIT
EL.-94.81
11'-0"
GROUTED PEA STONE
FACE ADVANCE, FT.
PEAGRAVEL
GROUT
675 FT
2240 FT
3145 FT
TIME IN DAYS
DATE
MARCH 10
MARCH 20
APRIL 1
APRIL 10
APRIL 20
MAY 1
JUNE 1
JUNE 18
JULY 1
UP
DOWN
δ, in.
MK-4 CROWN
MK-3 LEFT
MK-2 RIGHT
MK-1 INVERT
STRAIN (μ IN/IN)
1μ = 3.85 psi
STRAIN GAGE MEASUREMENTS LEGEND
+ = COMPRESSION
− = TENSION
• = IMBEDDED GAGE
x = SURFACE GAGE

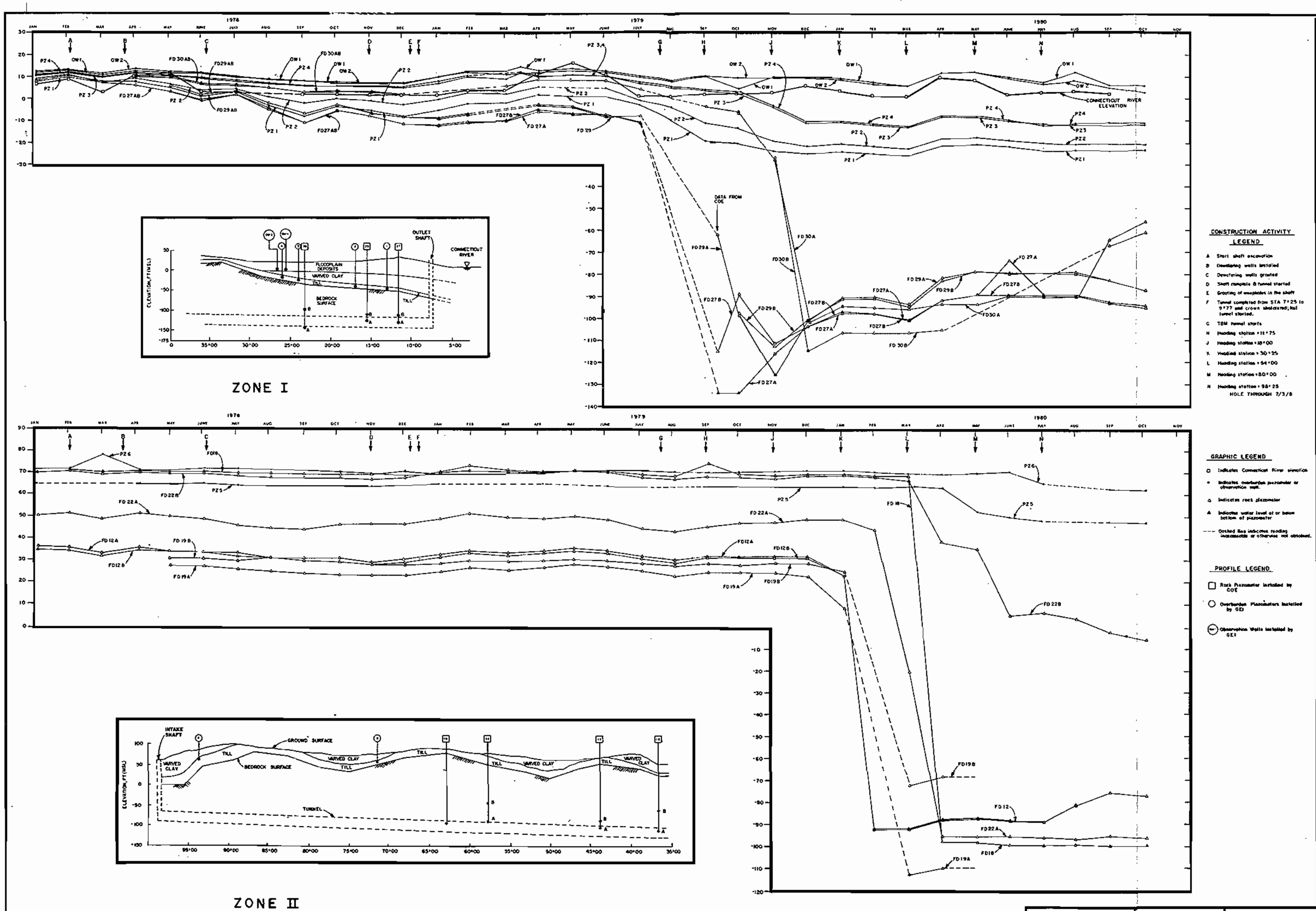
PLATE 27


MPBX NO. 8
GROUND SURFACE EL.+79.78
INSTRUMENT TEST SECTION
STA. 61+19
(RING NO.840)
TS-8
PARK RIVER AUXILIARY CONDUIT
DEPTH IN FEET BELOW GROUND SURFACE
ANCHOR (6) EL.-43.9
ANCHOR (5) EL.-67.9
ANCHOR (4) EL.-79.9
ANCHOR (3) EL.-85.9
ANCHOR (2) EL.-87.9
ANCHOR (1) EL.-89.9
BOTTOM OF BORING
EL. -90.2
3.02'
CROWN OF CONDUIT
EL. -93.22
FACE ADVANCE, FT.
INSTALLATION COMPLETE
INSTALLED PEASTONE
375 FT
1875 FT
2975 FT
3700 FT
TIME IN DAYS
DATE
MAR 20, 25
APRIL 5
APR 14
APRIL 29
MAY 6
MAY 14
JUNE 6
JULY 1
UP
DOWN
δ, in
MK-4 CROWN
MK-3 LEFT
MK-2 RIGHT
MK-1 INVERT
STRAIN (μ IN/IN)
1μ = 3.85 Psi
GROUTED PEA STONE
STRAIN GAGE MEASUREMENTS LEGEND
+ = COMPRESSION
− = TENSION
• = IMBEDDED GAGE
x = SURFACE GAGE


MPBX NO.9
GROUND SURFACE EL.+89.59
INSTRUMENT TEST SECTION
STA.-91+00
(RING NO.1324)
TS-9
PARK RIVER AUXILIARY CONDUIT
DEPTH IN FEET BELOW GROUND SURFACE
ANCHOR(6) EL.-28.8
118.39
ANCHOR (5) EL.-52.8
ANCHOR (4) EL.-60.8
150.39
ANCHOR (3) EL.-68.8
158.39
161.89 ANCHOR (2) EL.-72.3
ANCHOR (1) EL.-72.8
162.39
BOTTOM OF BORING
CROWN OF CONDUIT
EL.-75.4
EL.-76.6
1.1'
11'-0"
GROUTED PEA STONE
FACE ADVANCE, FT.
PEA GRAVEL
323 FT
GROUT
671 FT
TIME IN DAYS
DATE
APR.14
MAY 1
JUNE 1
6/9
6/16
6/26 JULY 1
JULY 17
δ, in
UP
DOWN
MK-4 CROWN
MK-3 LEFT
MK-2 RIGHT
MK-1 INVERT
STRAIN (μ IN/IN)
1μ = 3.85 psi
STRAIN GAGE MEASUREMENTS LEGEND
+ = COMPRESSION
− = TENSION
• = IMBEDDED GAGE
x = SURFACE GAGE



INSTRUMENT TEST SECTION
STA. 95+54
(RING NO. 1399)
TS-10
PARK RIVER AUXILIARY CONDUIT
MPBX NO.10
GROUND SURFACE EL 76.01
DEPTH IN FEET BELOW GROUND SURFACE
ANCHOR (6) EL.-26.0
ANCHOR (5) EL. -48.5
ANCHOR (4) EL. -60.5
ANCHOR (3) EL.- 65.0
ANCHOR (2) EL.-69.0
ANCHOR (1) EL.-71.0
BOTTOM OF BORING
EL.-71.5
CROWN OF CONDUIT
EL.-73.0
1.5'
11'-0"
GROUTED PEA STONE
FACE ADVANCE, FT.
217 FT
CONTACT GROUTED
DATE
JUNE 1
JULY 1
AUG. 1
δ, in
UP
DOWN
MK-4 CROWN
MK-3 LEFT
MK-2 RIGHT
MK-1 INVERT
STRAIN (μ IN/IN)
1μ = 3.85 Psi
STRAIN GAGE MEASUREMENTS LEGEND
+ = COMPRESSION
- = TENSION
• = IMBEDDED GAGE
x = SURFACE GAGE



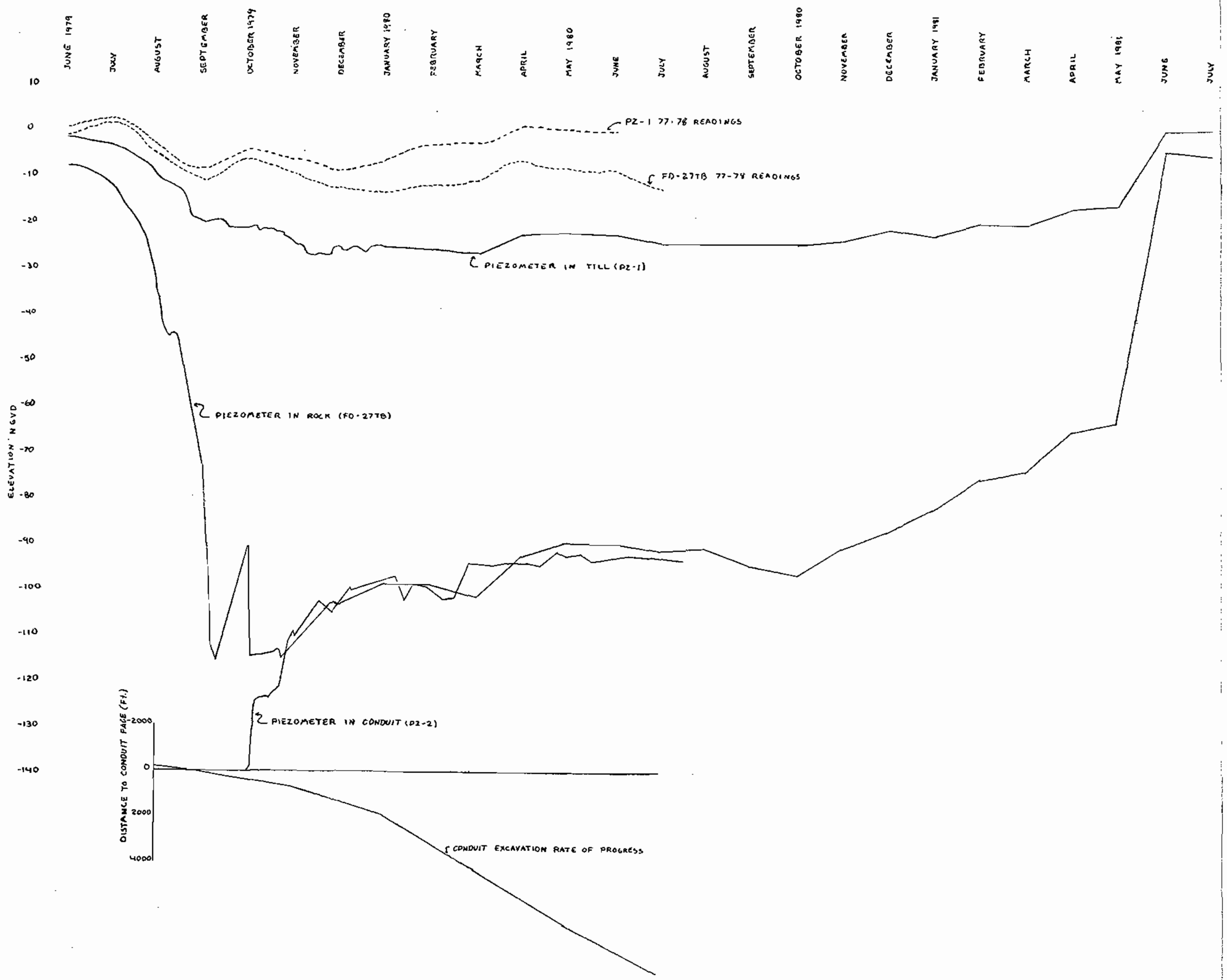
ROGER J. AU & SON, INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL HARTFORD, CT.	SURFACE PIEZOMETER AND OBSERVATION WELL DATA
GEOTECHNICAL ENGINEERS INC. WINCHESTER, MASSACHUSETTS	PROJECT 77382	OCT., 1980 FIG. F-1

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

PARK RIVER AUXILIARY CONDUIT

SURFACE PIEZOMETER AND OBSERVATION WELL DATA

PLATE 31

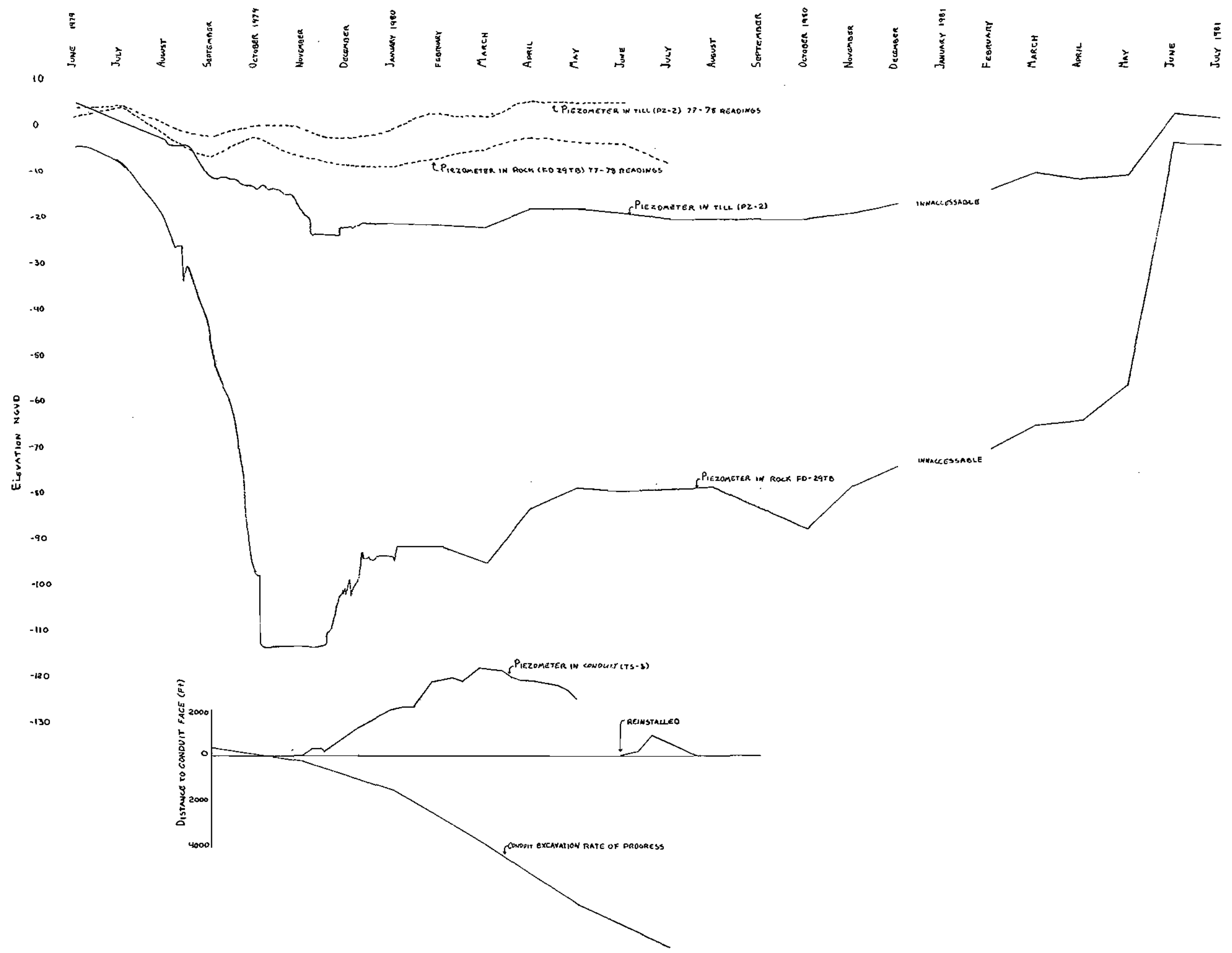


DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

PARK RIVER AUXILIARY CONDUIT

WATER LEVEL SUMMARY

TEST SECTION 2, RING 15, STA. 11+32

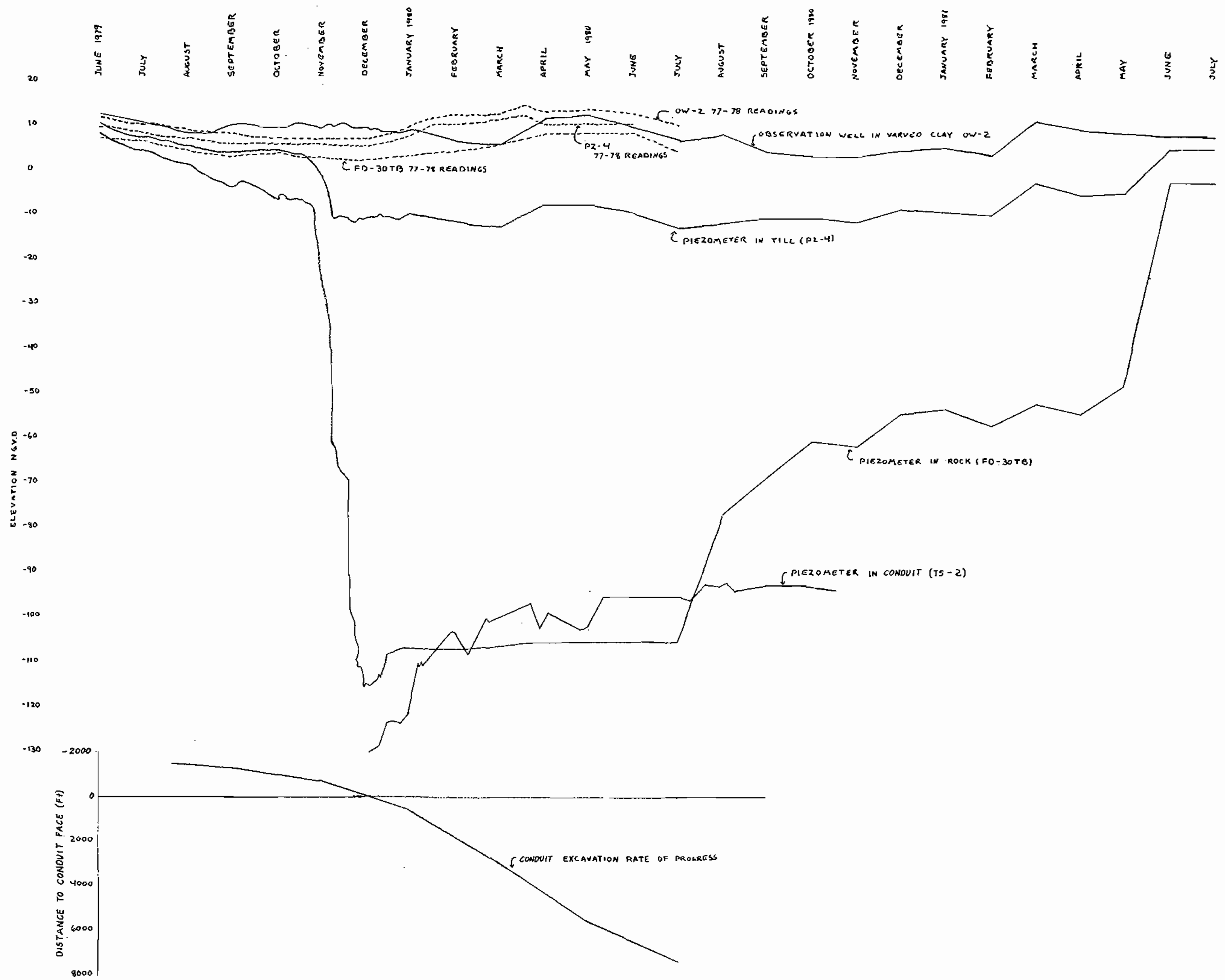


DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

PARK RIVER AUXILIARY CONDUIT

WATER LEVEL SUMMARY

TEST SECTION 3, RING 81, STA. 15+25



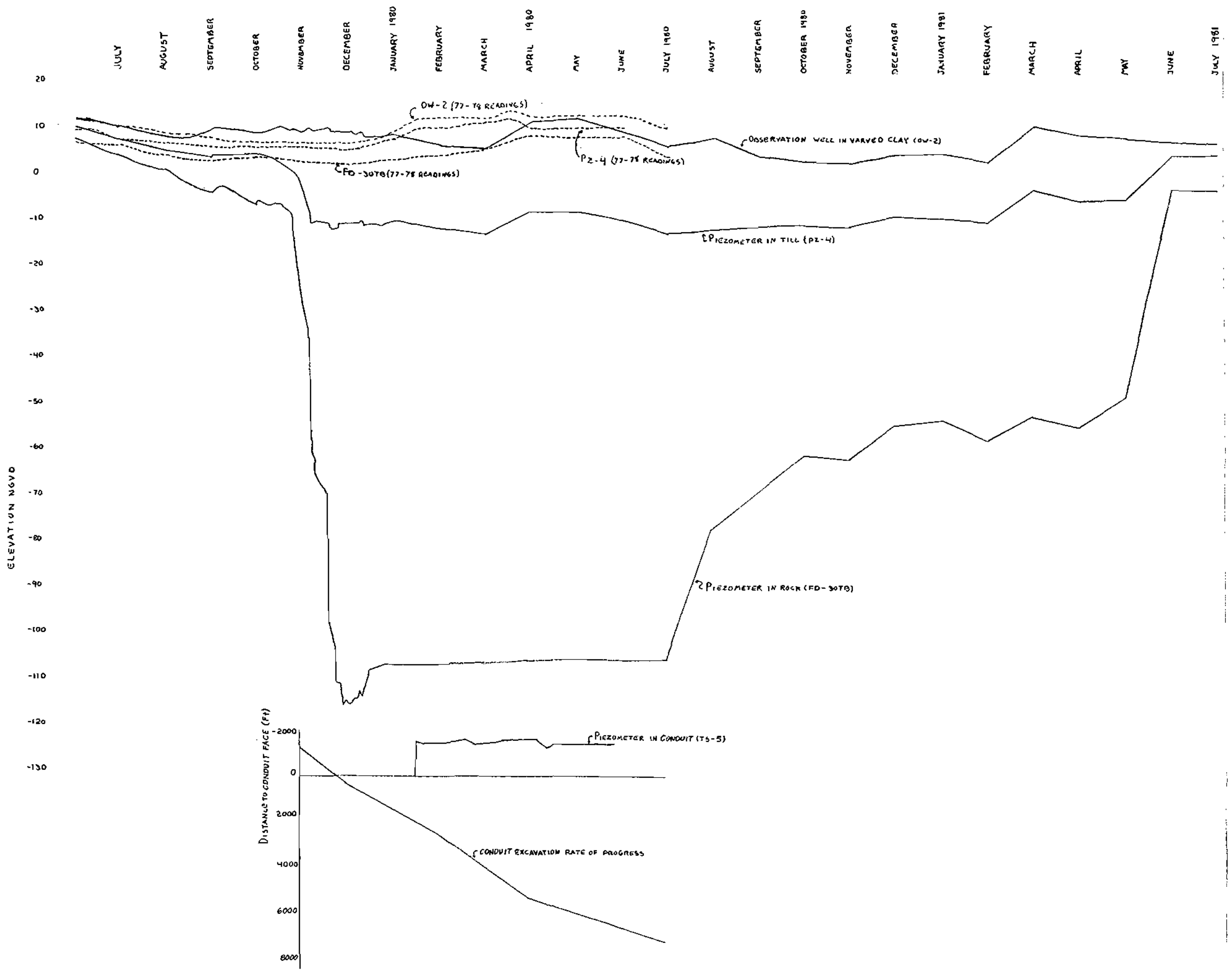
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

PARK RIVER AUXILIARY CONDUIT

WATER LEVEL SUMMARY

TEST SECTION 4, RING 225, STA. 27+00

PLATE 34

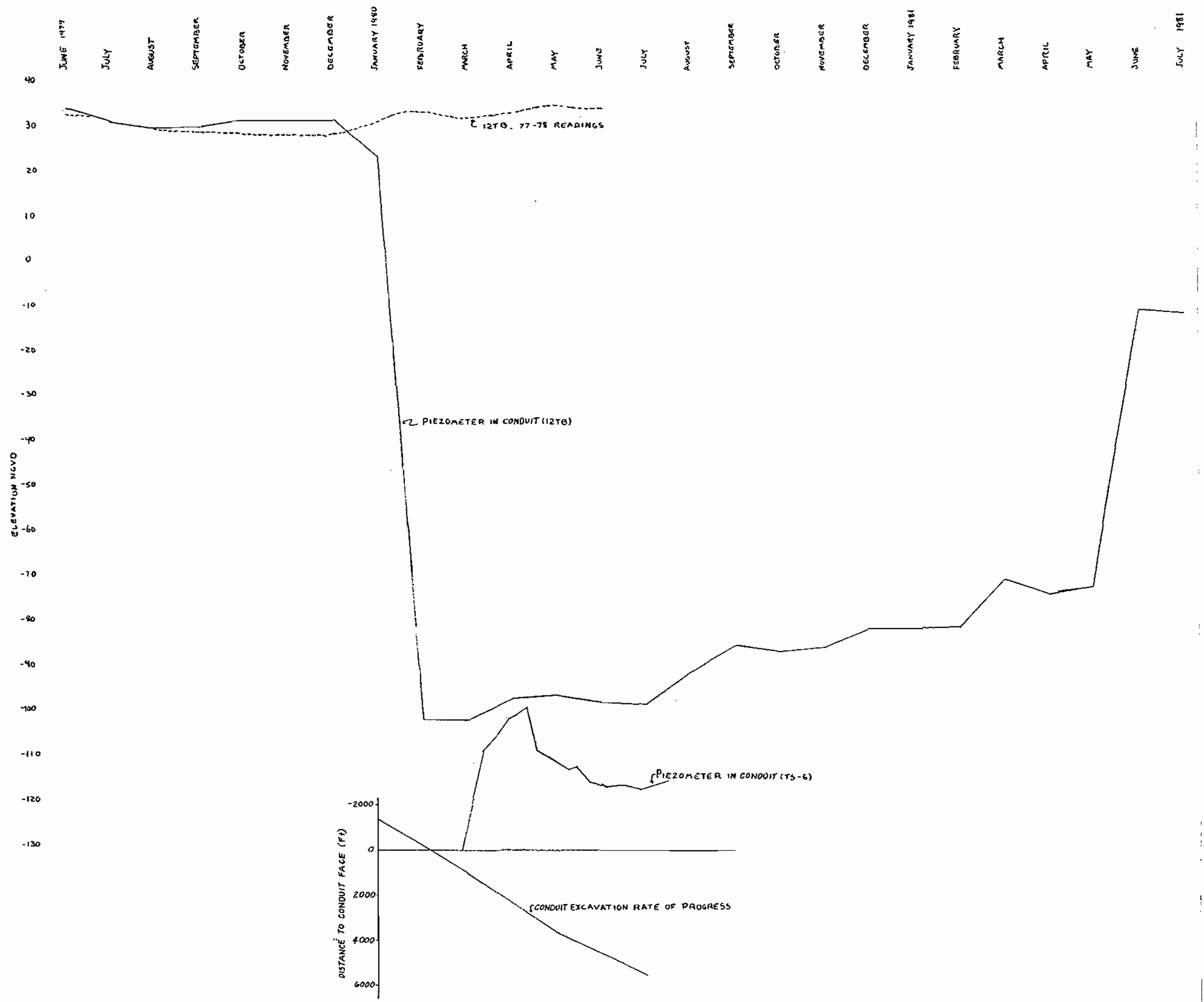


DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

PARK RIVER AUXILIARY CONDUIT

WATER LEVEL SUMMARY

TEST SECTION 5, RING 275, STA. 27+00

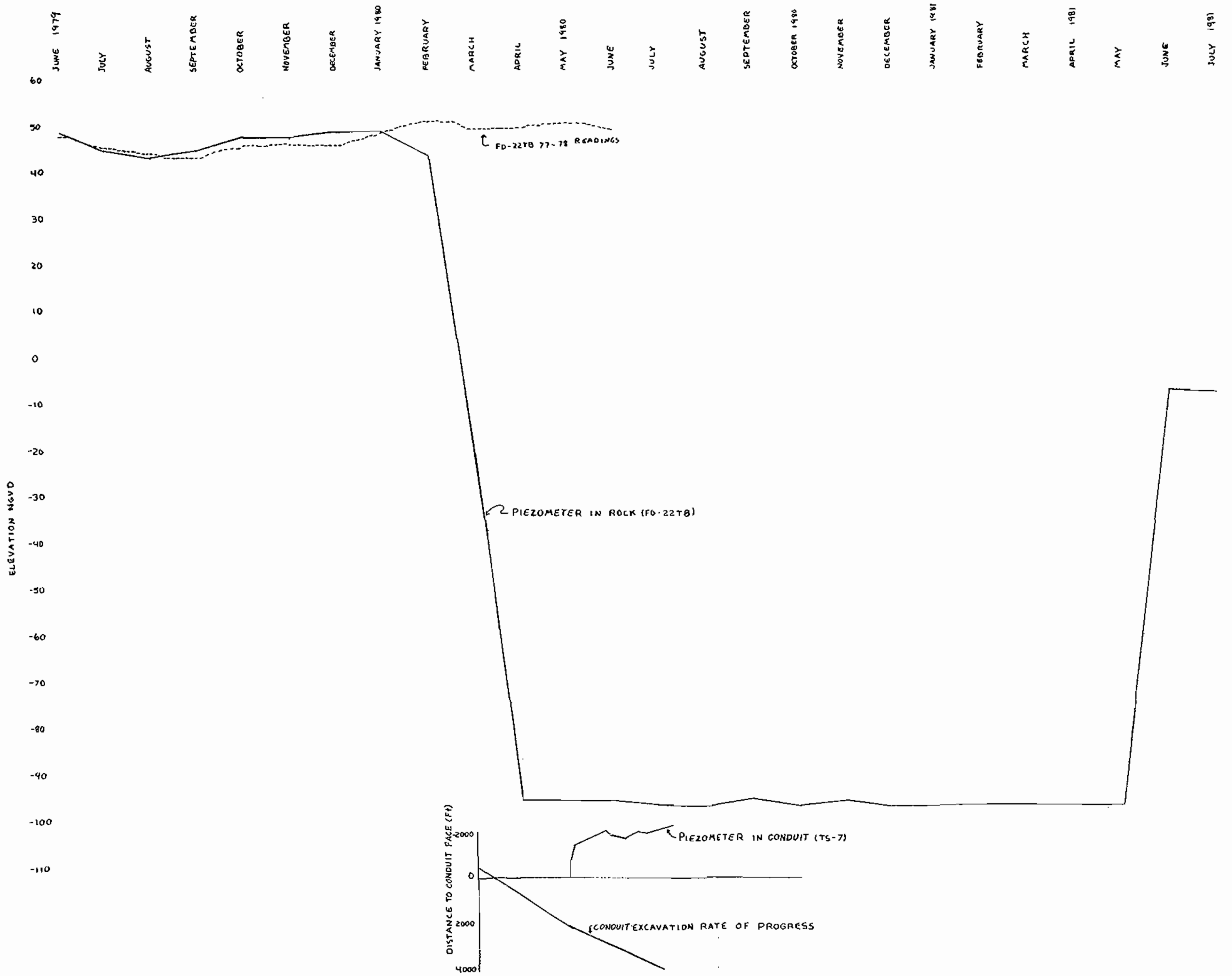


DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

PARK RIVER AUXILIARY CONDUIT

WATER LEVEL SUMMARY

TEST SECTION 2, RING 548, STA. 43+50



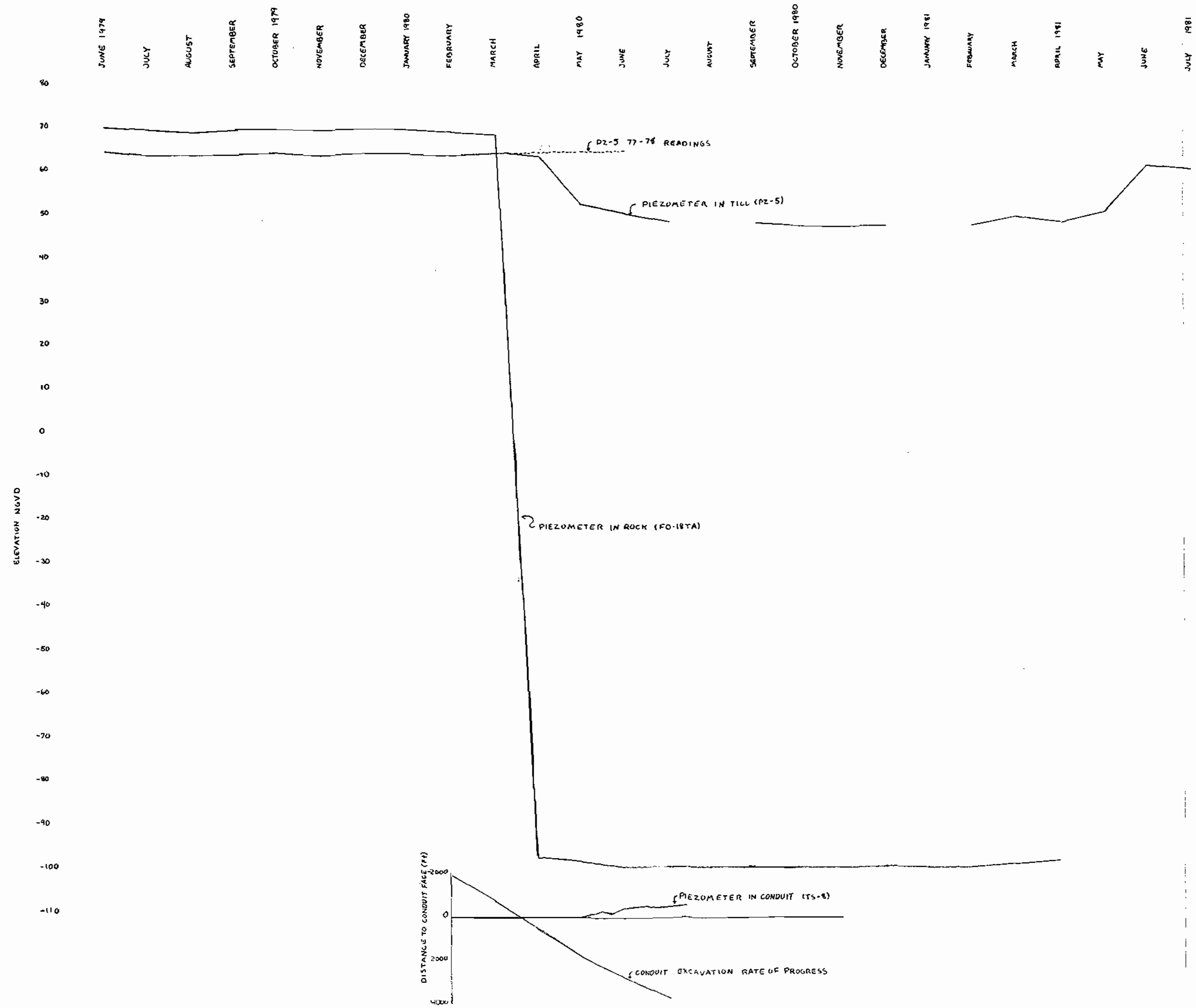
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

PARK RIVER AUXILIARY CONDUIT

WATER LEVEL SUMMARY

TEST SECTION 7, RING 797, STA. 58+55

PLATE 37

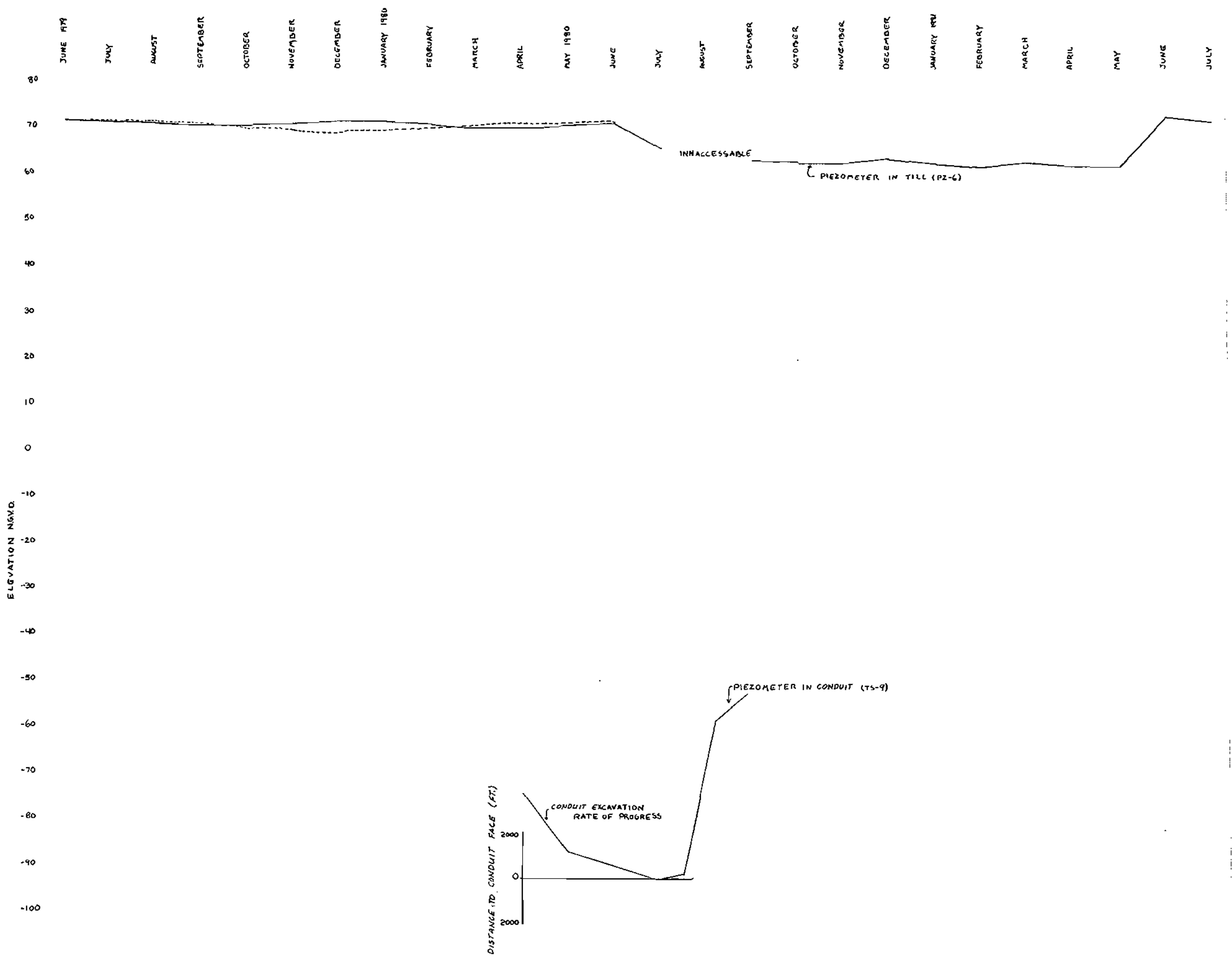


DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

PARK RIVER AUXILIARY CONDUIT

WATER LEVEL SUMMARY

TEST SECTION 8, RING 840, STA. 61+19

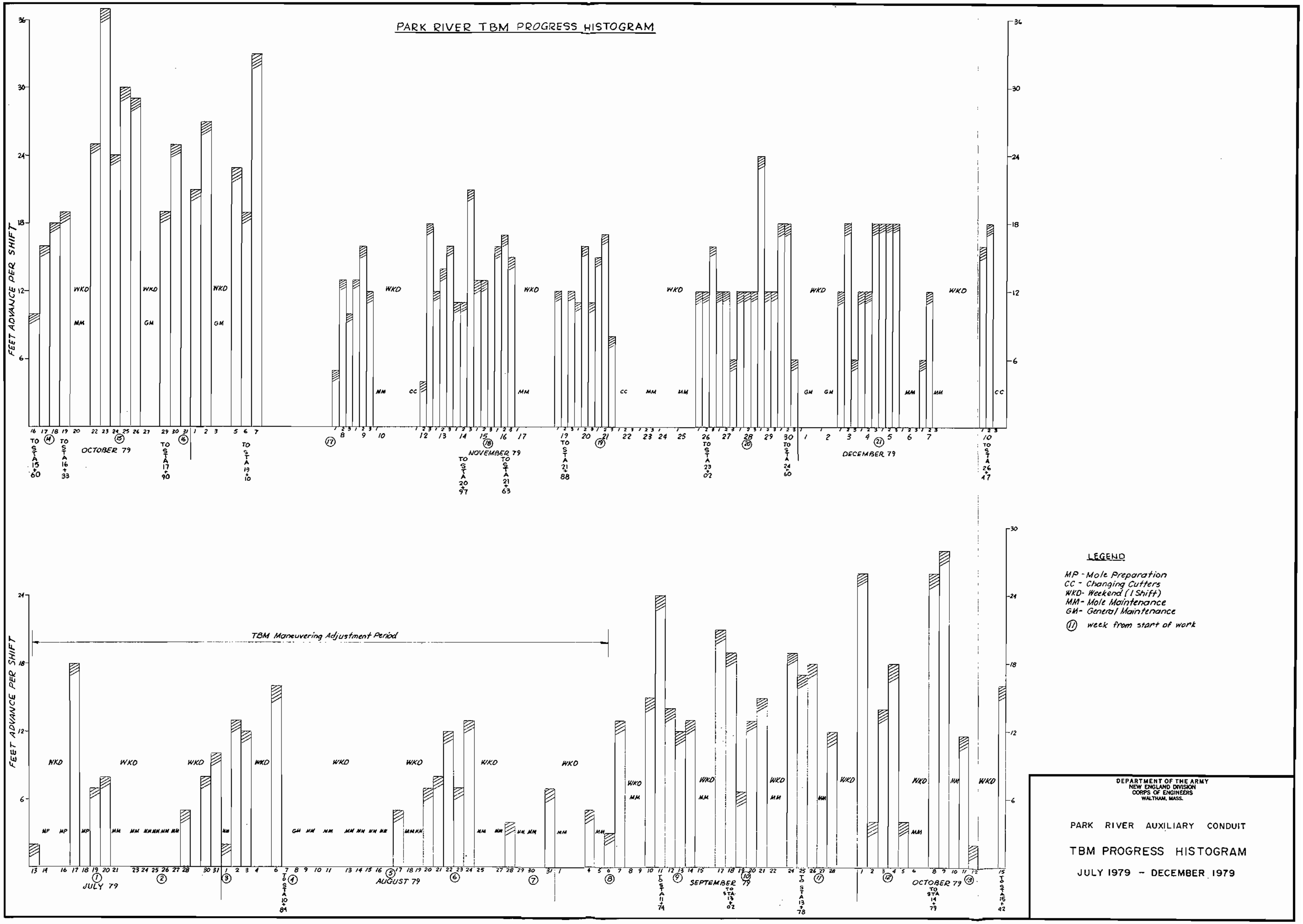


DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

PARK RIVER AUXILIARY CONDUIT

WATER LEVEL SUMMARY

TEST SECTION 4, RING 1394, STA. 11+00

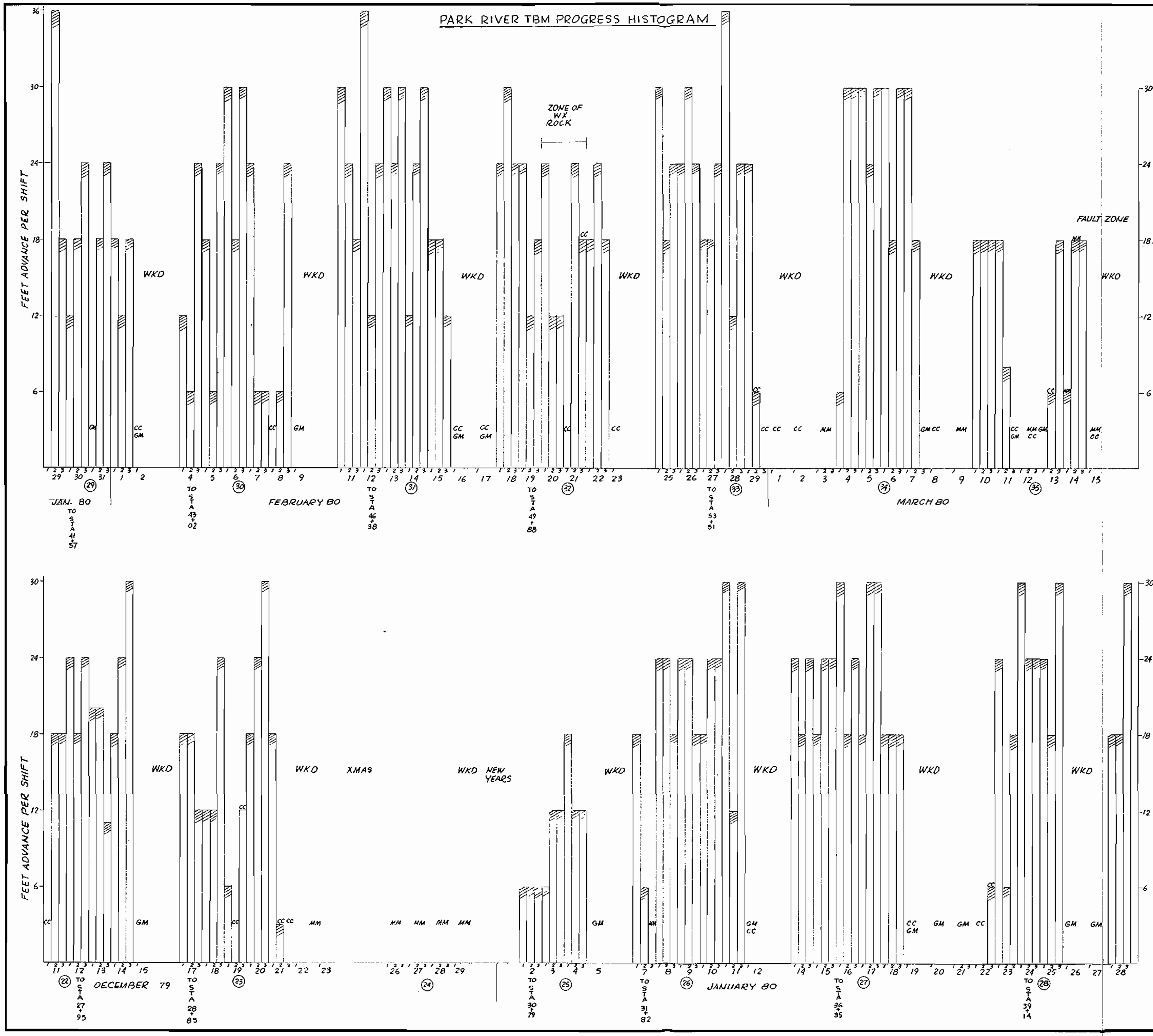


DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

PARK RIVER AUXILIARY CONDUIT

TBM PROGRESS HISTOGRAM

JULY 1979 - DECEMBER 1979



LEGEND

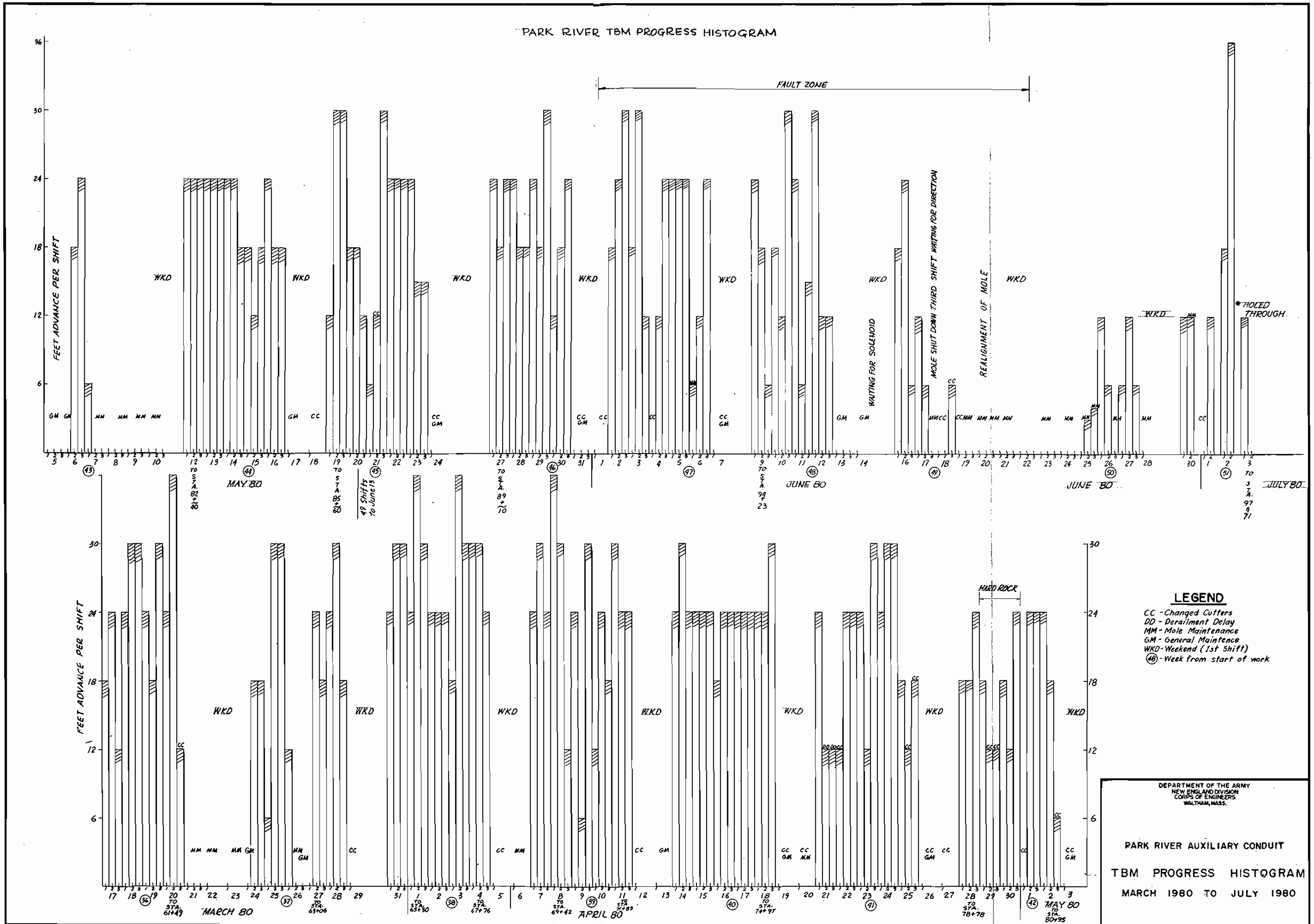
GM- General Maintenance
MM- Mole Maintenance
CC- Changing Cutters
WKD- Weekend (1 Shift)
(28) week from start of work

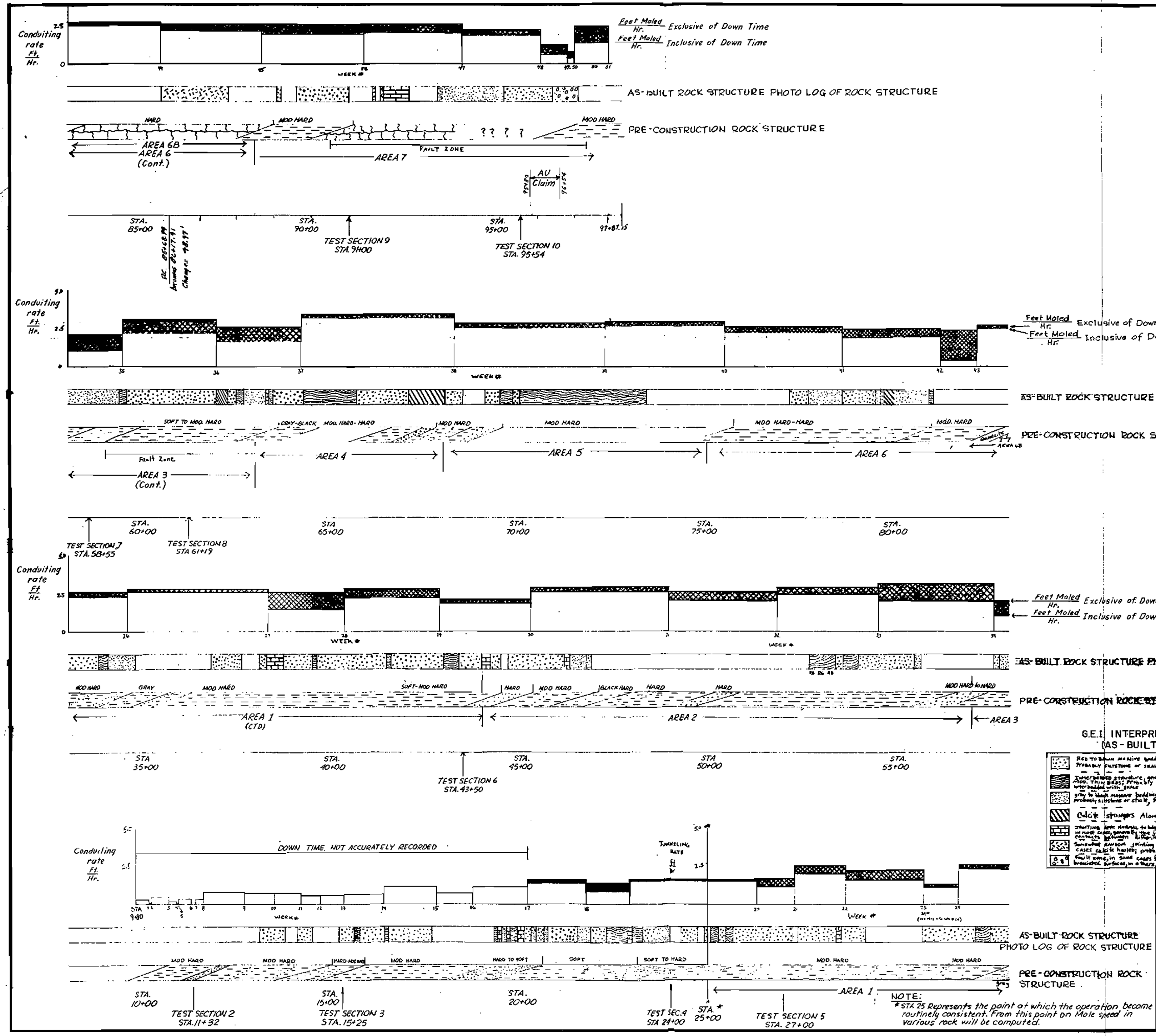
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

PARK RIVER AUXILIARY CONDUIT

TBM PROGRESS HISTOGRAM

DECEMBER 1979 – MARCH 1980


PARK RIVER TBM PROGRESS HISTOGRAM
FEET ADVANCE PER SHIFT
FAULT ZONE
WAITING FOR SOLENOID
MOLE SHUT DOWN THIRD SHIFT WAITING FOR DIRECTION
REALIGNMENT OF MOLE
HOLED THROUGH
HARD ROCK
WKD
MARCH 80
APRIL 80
MAY 80
JUNE 80
JULY 80
49 Shifts to June 13
LEGEND
CC - Changed Cutters
DD - Derailment Delay
MM - Mole Maintenance
GM - General Maintence
WKD - Weekend (1st Shift)
(48) - Week from start of work
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
PARK RIVER AUXILIARY CONDUIT
TBM PROGRESS HISTOGRAM
MARCH 1980 TO JULY 1980

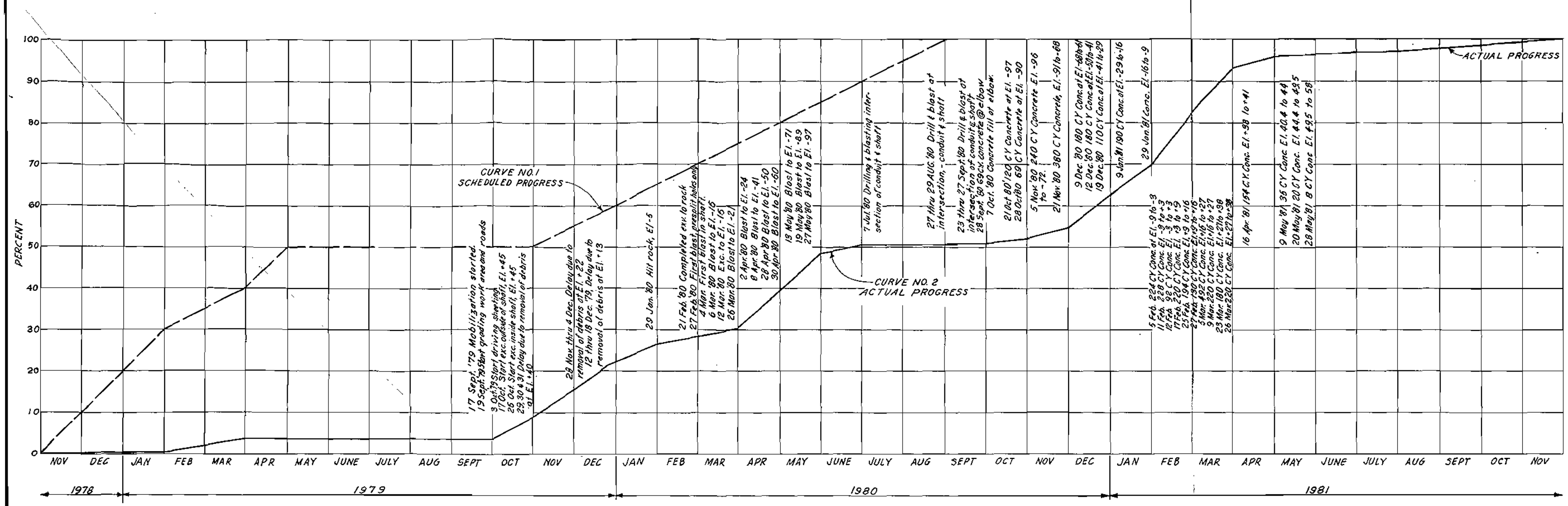


DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

PARK RIVER AUXILIARY CONDUIT

PARK RIVER ROCK CLASSIFICATION VS. MOLE ADVANCE RATE

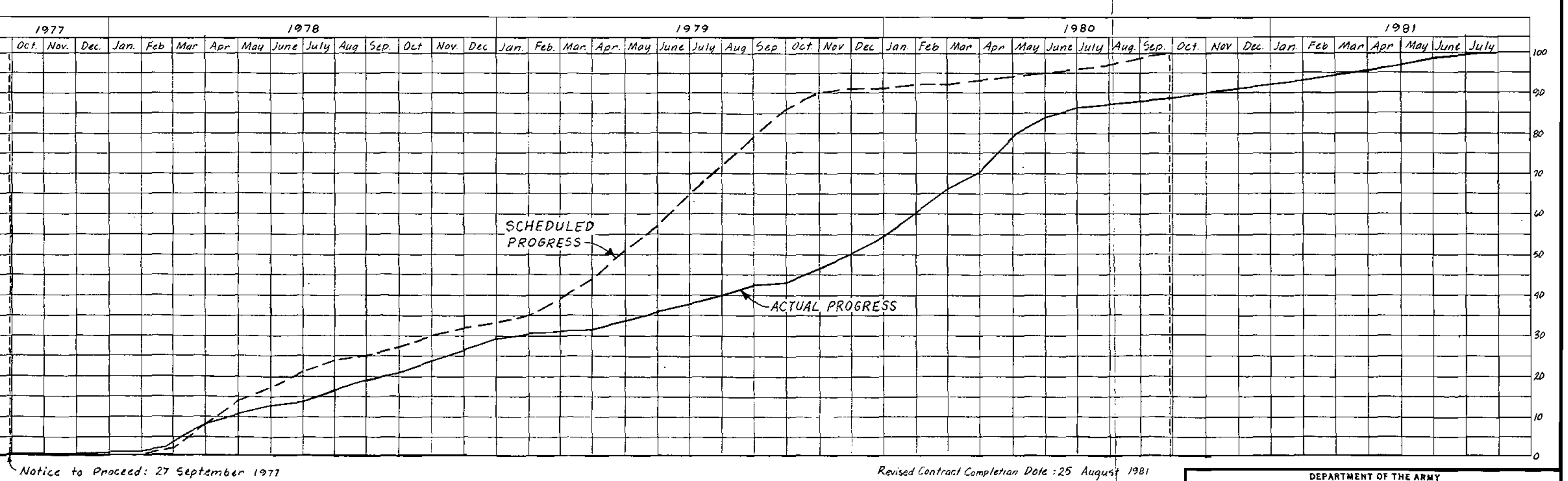
PLATE 43



INTAKE SHAFT

PRINCIPAL CONTRACT FEATURE	LINE ITEM	WT	ESTIMATED COST
JOB AS A WHOLE		100	$23,248,185 ($24,477,492)
2. Intake Structure & Shaft		6.45	1,500,000 *(1,584,891)
3. Outlet Structure & Shaft		17.21	4,000,000 *(4,191,245)
4. Removal of Bushnell Park Dike		0.65	150,000 *(157,565)
5. Broad Street Drainage Shaft		0.65	150,000 *(152,580)
9. Bituminous Concrete Pavement		0.72	168,000 *(173,730)
51. Mobilization & Preparatory Work		20.65	4,800,000 *(4,800,000)
53. Conduit Construction 53a. VE #1		50.39	11,715,840 *(12,306,787)
56. Conduit Instrumentation		1.50	350,000 *(350,000)
All Other Bid Items		1.78	414,345 *(760,703)

*( ) FINAL PAYMENT

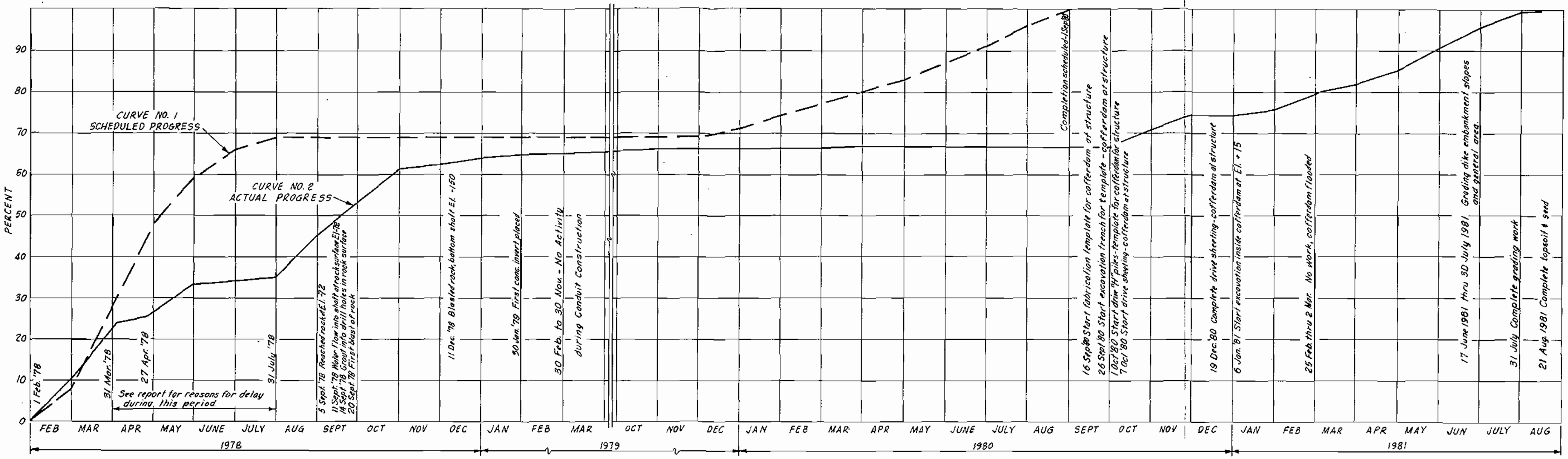


CONSTRUCTION PROGRESS CHART

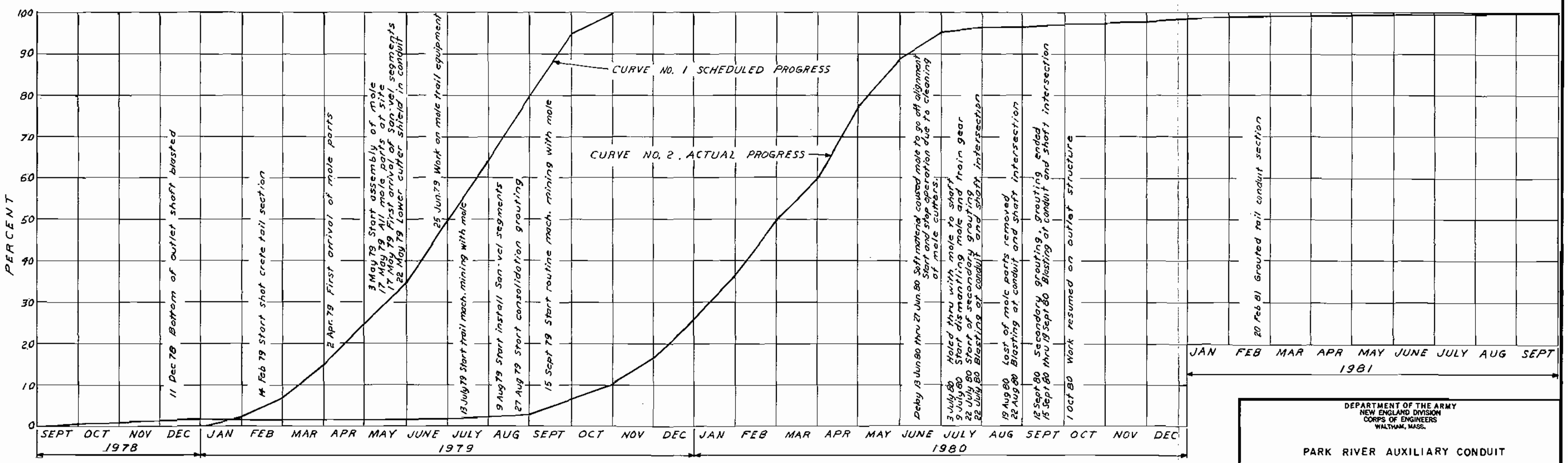
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

PARK RIVER AUXILIARY CONDUIT
ITEM NO. 2

INTAKE STRUCTURE & CONSTRUCTION PROGRESS CHART
SCHEDULED AND ACTUAL PROGRESS
(BASED ON EARNINGS)



OUTLET SHAFT



CONDUIT CONSTRUCTION

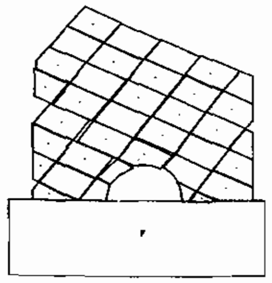
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

PARK RIVER AUXILIARY CONDUIT
ITEM NO. 53

OUTLET SHAFT AND
CONDUIT CONSTRUCTION
SCHEDULED AND ACTUAL PROGRESS
(BASED ON EARNINGS)

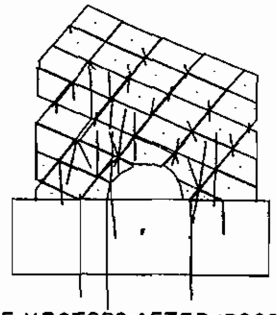
PLATE 45

13062 CYCLES DEFAULT MU-0.516



CONFIGURATION OF BLOCKS AFTER 13062 CYCLES (512 HOURS) WITHOUT PINNING, SHOWING SLIPPAGE OF 3 BLOCKS

13062 CYCLES DEFAULT MU-0.516



FORCE VECTORS AFTER 13062 CYCLES (512 HOURS) WITHOUT PINNING

NOTES ON ROCK STRUCTURE

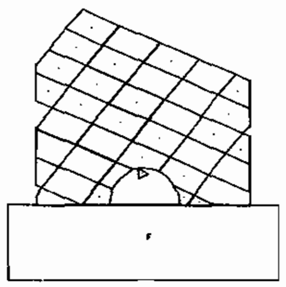
JOINT SPACINGS, ELEVATIONS, DIP AND STRIKE WERE DERIVED FROM THE MPBX-2 CORE SAMPLES.

ONE JOINT SET DIPS 30 DEGREES TO THE SOUTHEAST, HAS AN APPARENT DIP OF 22 DEGREES PROJECTED ONTO THIS NORTH-SOUTH SECTION, A SPACING OF 9.8 FEET AND A COEFICIENT OF FRICTION OF 0.516.

THE SECOND SET DIPS 60 DEGREES TO THE NORTHEAST, HAS AN APPARENT DIP OF 50 DEGREES, A SPACING OF 21 FEET, AND A COEFICIENT OF FRICTION OF 0.626.

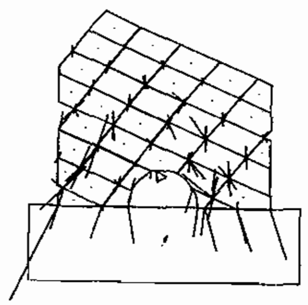
ONE COMPUTER UNIT OF FORCE EQUALS ROUGHLY TWO KIPS.

07712 CYCLES DEFAULT MU-0.516



CONFIGURATION OF BLOCKS AFTER 7712 CYCLES (302 HOURS) WITH A SIMULATED ROCKBOLT APPLYING 4 KIPS OF VERTICAL FORCE, SHOWING FAILURE STILL OCCURRING

07712 CYCLES DEFAULT MU-0.516



FORCE VECTORS AFTER 7712 CYCLES (302 HOURS) WITH 4 KIPS OF VERTICAL FORCE

07712 CYCLES DEFAULT MU-0.516

CENTROID COORDINATES
X CENTROID 476
Y CENTROID 325

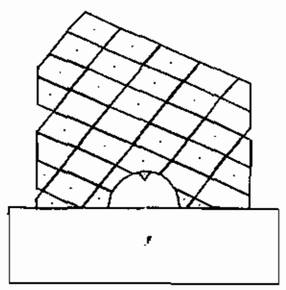
APPLIED LOADS
X LOAD +0
Y LOAD +5

X FORCE SUM +0
Y FORCE SUM -1
MOM. SUM +0

X VELOCITY -1530
Y VELOCITY -1217
ROT. VEL. -16383

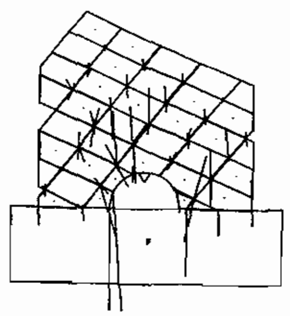
FORCES AND LOADS ON SIMULATED ROCKBOLT

08337 CYCLES DEFAULT MU-0.516



CONFIGURATION OF BLOCKS AFTER 8337 CYCLES (327 HOURS) WITH SIMULATED ROCKBOLT APPLYING 6 KIPS OF VERTICAL FORCE, SHOWING A GENERAL UPLIFT OF BLOCKS

08337 CYCLES DEFAULT MU-0.516



FORCE VECTORS AFTER 8337 CYCLES (327 HOURS) WITH 6 KIPS OF VERTICAL FORCE

08337 CYCLES DEFAULT MU-0.516

CENTROID COORDINATES
X CENTROID 479
Y CENTROID 329

APPLIED LOADS
X LOAD +0
Y LOAD +6

X FORCE SUM +0
Y FORCE SUM +3
MOM. SUM +0

FORCES AND LOADS ON SIMULATED ROCKBOLT

SEQUENCE 1 - CROSS SECTION 2

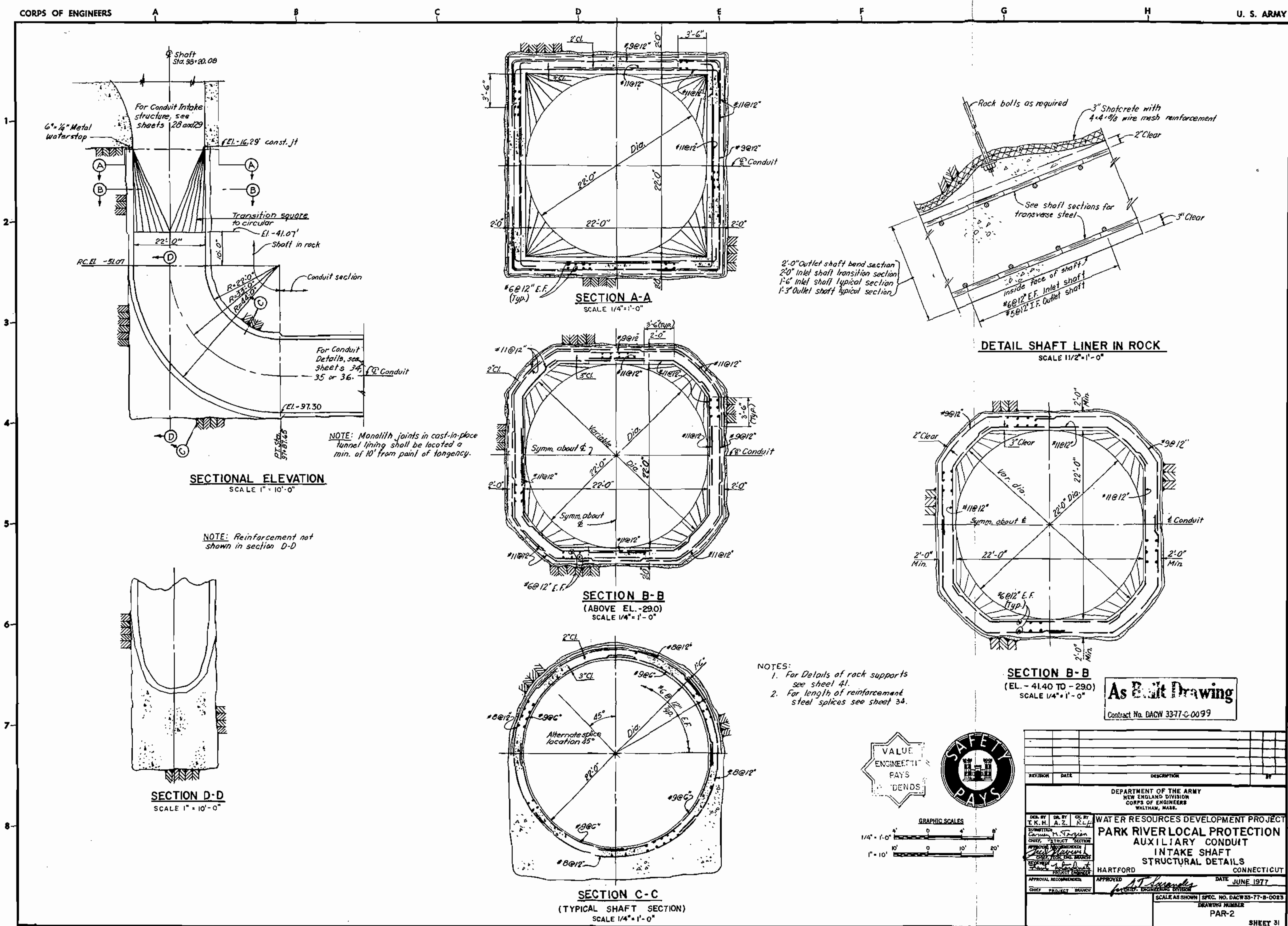
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

PARK RIVER AUXILIARY CONDUIT

ROCK LOAD DIAGRAM

INTERACTIVE GRAPHIC METHOD

TEST SECTION 2


CORPS OF ENGINEERS
U. S. ARMY
SECTIONAL ELEVATION
SCALE 1" = 10'-0"
℄ Shaft Sta. 98+20.08
For Conduit Intake structure, see sheets 28 and 29
6"x 1/4" Metal waterstop
EL. -16.29' const. jt
Transition square to circular
EL. -41.07'
Shaft in rock
P.C. EL. -51.07
Conduit section
For Conduit Details, see sheets 34, 35 or 36.
℄ Conduit
EL. -97.30
NOTE: Monolith joints in cast-in-place tunnel lining shall be located a min. of 10' from point of tangency.
NOTE: Reinforcement not shown in section D-D
SECTION D-D
SCALE 1" = 10'-0"
SECTION A-A
SCALE 1/4" = 1'-0"
SECTION B-B
(ABOVE EL. -29.0)
SCALE 1/4" = 1'-0"
SECTION C-C
(TYPICAL SHAFT SECTION)
SCALE 1/4" = 1'-0"
DETAIL SHAFT LINER IN ROCK
SCALE 1 1/2" = 1'-0"
Rock bolts as required
3" Shotcrete with 4x4-8/8 wire mesh reinforcement
2" Clear
3" Clear
See shaft sections for transverse steel
2'-0" Outlet shaft bend section
2'-0" Inlet shaft transition section
1'-6" Inlet shaft typical section
1'-3" Outlet shaft typical section
Inside face of shaft
#6@12" E.F. Inlet shaft
#5@12" I.F. Outlet shaft
SECTION B-B
(EL. -41.40 TO -29.0)
SCALE 1/4" = 1'-0"
NOTES:
1. For Details of rock supports see sheet 41.
2. For length of reinforcement steel splices see sheet 34.
As Built Drawing
Contract No. DACW 33-77-C-0099
VALUE ENGINEERING PAYS DIVIDENDS
SAFETY PAYS
GRAPHIC SCALES
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT
INTAKE SHAFT
STRUCTURAL DETAILS
HARTFORD
CONNECTICUT
DATE JUNE 1977
SCALE AS SHOWN
SPEC. NO. DACW33-77-B-0023
DRAWING NUMBER
PAR-2
SHEET 31